PRODUCTION OF MEDIUM TO LOW STRENGTH CONCRETE UTILIZING INDIGENOUS WASTE PRODUCTS

BY

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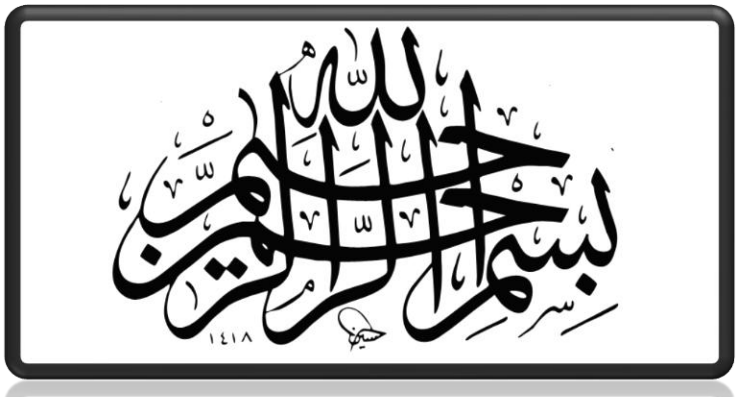

IN THE NAME OF ALLAH, MOST GRACIOUS, MOST MERCIFUL

Dedicated To

My Parents,

Syed Ferhatullah Hussainy,

&

Syed Amanullah Hussainy

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All praise is due only to **ALLAH** subhana wa ta' aala, the sustainer of the worlds, the most compassionate, the most merciful for bestowing me patience, health and knowledge to complete this work successfully. May the peace and blessings of Allah Subhanahu wa ta'aala be upon Prophet Mohammed (Sal allahu alahi wa sallam).

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THESIS ABSTRACT (ENGLISH)

NAME: **SYED KHAJA NAJAMDDIN**

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Several by-products of industrial processes, such as cement kiln dust (CKD), limestone powder (LSP), electric arc furnace dust (EAFD), oil ash (OA), cause environmental and storage problems to the local industry. Their utilization in concrete, as a replacement of cement, would be beneficial from both economic and environmental perspectives. A significant quantity of cement could be replaced to produce low strength concrete while replacement of a lesser quantity would result in medium to normal strength concrete. In both cases, however, there would be significant economic and environmental benefits.

This study was conducted to investigate the possibility of producing low to medium strength concrete utilizing local waste products, such as CKD, EAFD, OA, LSP and Natural Pozzolan (NP). The mechanical, electrical, and thermal properties of concretes produced utilizing the selected local waste materials were assessed. Results of this investigation show that 0 to 20% of the selected waste materials can be utilized to produce high to medium strength concrete while 50 to 60% of the selected waste materials can be used to produce medium to low strength concrete. Concretes with 5% CKD, LSP OA, or EAFD or 10% NP (replacement of cement) exhibited superior mechanical properties and durability characteristics. Consequently, the use of these materials will lead to a reduction in the consumption of cement and the greenhouse gas emission.

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ملخص الرسالة

الإسم: سيد خاجة نجامودين

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و مسحوق الحجر الجيري(CKD)إن العديد من النواتج الثانوية للعمليات الصناعية كغبار الإسمنت وتسبب العديد من مشاكل بيئية و مشاكل في (OA))EAFDبالاضافة إلى رماد النفط ((LSP) تخزينها للصناعة المحلية ، إن إستغلال هذه المواد في الخرسانة من خلال استبدال الإسمنت سيحقق منافع اقتصادية وبيئية أيضا. إن كمية مؤثرة من الإسمنت يمكن إستبدالها بهذه المواد من أجل الحصول على خرسانة ضعيفة القوة بينما أن إستبدال كمية قليلة تؤدي للحصول على خرسانة متوسطة القوة. في كلتا الحالتين يمكن الحصول على منفعة إقتصادية وبيئية مؤثرة.

تهتم هذه الدراسة بدراسة إمكانية الحصول على خرسنة ضعيفة إلى متوسطة القوة من خلال إستغلال . أن الخصائص (NP)البوزولان الطبيعي CKD, EAFD, OA, LSPنفايات محلية مثل تبين أنه يمكن ,الميكانيكية والكهربائية والحرارية للإسمنت أثرت في إختيار النفايات المحلية استخدامها في 0-20 ٪ من نفايات المواد المختارة لإنتاج الخرسانة عالية لقوة متوسطة في حين يمكن استخدام 50 إلى 60 ٪ من نفايات المواد المختارة لإنتاج الخرسانة المتوسطة لقوة منخفضة. و (استبدال الأسمنت) متفوقة 10% NP EAFD او ,CKD, LSP OA 5%خرسانة مع وبالتالي ، فإن استعمال هذه المواد تؤدي إلى انخفاض في الخواص الميكانيكية وخصائص المتانة استهلاك الاسمنت وانبعاث غازات الاحتباس الحراري

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CHAPTER 1

INTRODUCTION

1.1 Utilization of Waste Materials in Concrete

Cement is a substance that is finely pulverized, dry and acts as binder to glue other materials together as a result of hydration after addition of water. The most important use of cement is the production of concrete. The annual production of cement in the world is about 3,300 million tons. Cement manufacture contributes greenhouse gases both directly through the production of carbon dioxide when calcium carbonate is burnt in the kilns, producing lime and carbon dioxide (CO_2) and also indirectly through the use of energy in the kiln, particularly if the energy is sourced from fossil fuels [1]. The "greenhouse effect" is a process by which the earth is becoming warmer with time. The earth is bathed in sunlight, some of it is reflected back into space and some is absorbed. If the absorption is not matched by radiation back into space, the earth will get warmer until the intensity of that radiation matches the incoming sunlight. Some atmospheric gases absorb outward infrared radiation, warming the atmosphere. Carbon dioxide is one of these gases; so are methane, nitrous oxide, and the chlorofluorocarbons (CFCs). The concentrations of these gases are increasing, with the result that the earth is absorbing more sunlight and getting warmer [2]. The cement industry produces 5% of global man made CO_2 emissions, of which 50% is from the chemical process and 40% from burning fuel [3]. The volume of CO_2 emitted by the cement industry is nearly 900 kg for every 1000 kg of cement produced [4].

The Saudi building construction industry has seen exceptional growth in recent years with the construction contract awards likely to grow to $39,138 million in 2012, in spite of the global financial crisis. The construction sector in Saudi Arabia is the largest and fastest growing market in the Arabian Gulf region. Ongoing construction projects in the Gulf are valued at $1.9 trillion. Riyadh allocates more than $695 billion to be spent on 687 projects before the end of 2015 [5]. Hence, this will increase the consumption of cement tremendously. The total annual production of Portland cement in Saudi Arabia has increased from 0.66 million tons in 1960 to more than 14.3 million tons in 1995 and was expected to reach 53 million tons in 2010 [6,7].

Though cement is the main source of strength in concrete, there is a need to decrease its quantity used by the construction industry in order to decrease the greenhouse gases and to conserve energy and mineral resources. The concrete elements used in non-structural applications such as pavements, blinding concrete, masonry blocks, fill material, electrical ducts, etc., do not require high strength. For such applications, blended cements developed utilizing locally available waste materials, such as cement kiln dust (CKD) produced during the production of cement, electric arc furnace dust (EAFD) produced during the production of steel, limestone powder (LSP) produced during the crushing of large size boulders to produce small size aggregates, oil ash (OA) produced when oil is burnt in the power generation plants and natural pozzolan (NP), etc., would produce the required "necessary" medium to low strength concrete.

CKD is an important by-product material of the cement manufacturing process. It is generated from burning of the raw materials in a rotary kiln to produce clinker. It is a fine

grained, solid and highly alkaline material. In general, CKD is a very heterogeneous mix both by chemistry and particulate size that are dependent on the raw materials, fuels, kiln pyro-processing type, overall equipment layout, and type of cement being manufactured [8]. It consists primarily of calcium carbonate and silicon dioxide that are similar to the cement kiln raw feed, but the amount of alkalis, chlorides and sulfates are considerably more in the CKD [9]. However, selected quantity of CKD could be utilized to produce medium to low strength concrete that could be utilized for non-structural purposes.

Dust, often denoted as EAFD emanating from an electric arc furnace, is collected into bag filters during the steel making process. This fine dust consists mostly of iron oxide and zinc oxide [10]. Other constituents include: oxides of calcium, magnesium, silicon, etc. EAFD is considered as a hazardous material by the Environmental Protection Agency (EPA) and, therefore, it needs to be disposed off after stabilization [10]. The world steel industry spends approximately $50 to $250 per ton to stabilize EAFD for landfill or for zinc recovery [10]. Initial studies conducted at King Fahd University of Petroleum and Minerals (KFUPM) have indicated that EAFD could be utilized as a retarder in concrete [10]. However, it is possible to utilize it as a filler and/or a cementitious material.

LSP is obtained during the crushing of carbonate rocks, which are considered as the primary source of coarse aggregates in the Eastern Province of the Kingdom [11]. LSP could be utilized as a replacement of cement or as a filler. The addition of limestone dust reduces the initial and final setting time, as well as porosity, whereas the free lime and combined water increase with increasing limestone content. Further, the addition of LSP enhances the rate of hydration as determined by the combined water contents [12]. Limestone filler affects the

crystallization nucleus for the precipitation of calcium hydroxide. These effects produce an acceleration of the hydration of cement grains [13]. It could also be utilized as a viscosity enhancer in the production of self-consolidated concrete [14].

OA is generated during the burning of oil in power generation plants. It is a very fine ash and most of it passes ASTM No. 200 sieve. Oil ash consists of inorganic substances, such as SiO_2, Fe_2O_3, and Al_2O_3 with 70 to 80% of unburned carbon [15]. It is presently disposed off as a waste material, posing environmental and storage problems. Initial screening tests conducted at KFUPM indicated that the pozzolanic activity of 5 to 20% OA cement mortar was much less than that of control concrete containing 100% cement [16]. The chemical characteristics of the fuel oil ash generated at a power plant differ significantly from that of coal fly ash. The carbon content of heavy fuel oil fly ash is about 95% while that of coal fly ash generally ranges between 20% and 50%. Toxic heavy metals, such as vanadium (2.08% as V_2O_5) and nickel (0.37% as NiO) are also present in the heavy fuel fly ash. The high carbon content and presence of toxic heavy metals suggested that this fuel oil fly ash be considered as a hazardous respirable dust that demands careful handling during study the possibility of utilizing OA in producing moderate to low strength concrete [17].

Natural Pozzolan is a raw or calcined natural material that has pozzolanic properties. It is one of the oldest materials used for construction purposes by blending it with lime. Its sources are volcanic ash or pumice, shales, tuffs and some diatomaceous earths [18]. Initial studies conducted at KFUPM have shown that the pozzolanic activity of this material is low and as such it does merely meet the ASTM C 618 requirements as a pozzolanic material [16].

Furthermore, it could be used as a filler material for producing medium to low strength concrete.

The above-mentioned industrial waste materials are abundantly available in the Kingdom of Saudi Arabia and they are rarely used in the construction industry. They are normally deposited in landfills. This causes environmental problems as well as contributing to large dust content in the air. In addition, there is a strong desire to reduce the consumption of cement through the effective utilization of industrial waste materials in order to decrease greenhouse effect and environmental problems.

1.2 Significance of This Research

The "fabricated" material that is most widely used in the world is concrete and cement is the principal ingredient in concrete, which means that more cement will be produced. However, cement manufacture consumes a lot of the oil resources in the Kingdom in order to generate the necessary energy for its production. Further, it is a source of greenhouse gas emission. In future, there will be increasing demand for concrete worldwide necessating the use of alternative cementitious materials. Research has been conducted at KFUPM on the use of fly ash, silica fume, blast furnace slag, metakaolin in concrete. The results of those studies [19-23] have shown that these materials tend to improve the mechanical properties and durability characteristics of concrete. However, most of these materials are not available locally and they have to be imported from other countries thereby leading to increase in the cost. Therefore, the utilization of indigenous waste materials is a noble task that will certainly lead to a reduction in the greenhouse gas emissions. Further, the usage of these waste and cheap

materials in concrete with medium to low strength will certainly reduce the cost of this important building material. The medium to low strength concrete incorporating waste materials can be utilized in non-structural applications, such as pavements, blinding concrete, masonry blocks, or as fill materials in electrical ducts, etc. Hence, there is a strong desire to utilize the locally available waste materials, such as CKD, LSP, EAFD, OA, NP, etc., in concrete to overcome environmental problems and produce a cheaper building material.

1.3 Objectives

The general objective of this study is to assess the possibility of producing medium to low strength concretes utilizing the locally available waste materials such as, CKD, LSP, EAFD, OA and NP. The specific objectives are the following:

1. To produce medium to low strength concrete utilizing the locally available waste materials,

2. To ascertain the mechanical, thermal, and electrical properties of medium to low strength concretes developed utilizing the locally available waste materials, and

3. To provide recommendations on the avenues for usage of concrete mixtures developed utilizing the locally available waste materials.

CHAPTER 2

LITERATURE REVIEW

2.1 Waste Materials Available in the Kingdom of Saudi Arabia

Several waste materials are generated during the industrial processes in the Kingdom of Saudi Arabia. Some of these materials and their characteristics are discussed in the following sub-sections.

2.1.1 Cement Kiln Dust (CKD)

Large quantities of CKD are produced during the manufacture of cement clinker by the dry process. The current (i.e. 2010) annual production of cement in the Kingdom is 53 million tons [7]. For each ton of clinker, a typical kiln generates around 6 to 7% ton of CKD [6]. Therefore, the generation of CKD in the Kingdom is approximately 3.5 million tons per year. The production of CKD in the Arabian Cement Company Ltd. (ACCL), Jeddah, is about 1000 tons per day, a portion of which is recycled into the kiln and a small portion is used by contractors and the rest is disposed of in landfills [24]. The siliceous raw materials of cement are sources of alkali salts, which include potassium and sodium feldspars in CKD [25]. The major reasons for CKD to be removed from the process are, first, the quality of clinker must be maintained without increasing the level of alkalis, chlorides, and/or sulfates. The other reason is that, CKD must be removed to maintain stability of kiln, otherwise, volatiles at high concentrations in the kiln can cause severe material buildup and this leads to the production

loss, blockage and even to shutting down of the kiln [26]. As CKD is a very fine powder, it is very difficult to handle and the cost of its disposal is very high and results in environmental pollution.

CKD that is categorized as waste can be used in many applications that include the following [27]:

- Agriculture: potash/lime source and animal feed.
- Civil engineering: fill, soil stabilization, fly ash stabilization, and blacktop filler.
- Building materials: lightweight aggregates, blocks, low strength concrete, and masonry cement.
- Sewage and water treatment: coagulation aid and sludge stabilization.
- Pollution control: sulphur absorbent, waste treatment, and solidification.

Some typical beneficial utilization of CKD are listed in Table 2.1. As detailed in the table, CKD has been mostly utilized in the building industry.

Table 2-1: Beneficial Utilization of CKD for 2006 year [28].

Avenues of utilization	**Quantity of CKD beneficially reused, Metric tons**	**Percentage (%) of total**
Soil / Clay Stabilization / Consolidation	533,365	45.98
Waste Stabilization / Solidification	213,675	18.42
Cement Additive / Blending	183,228	15.80
Mine Reclamation	152,756	13.17
Agricultural Soil Amendment	33,546	2.89
Sanitary Landfill Liner / Cover Material	15,042	1.30
Wastewater Neutralization / Stabilization	12,302	1.06
Pavement Manufacturing	12,066	1.04
Concrete Products	374	0.03
Others	3,657	0.32
Total	**1,160,011**	**100**

Note* – From PCA member company survey for 2006

Al-Harthy et al. [29] investigated the utilization of CKD as a cementitious material in concrete and mortar. The compressive strength of concrete with 0%, 5%, 10%, 15%, 20%, 25%, and 30% CKD was determined after 3, 7, and 28 days. Three different w/c ratios, 0.5, 0.6, and 0.7 by weight, were considered. The authors reported that the replacement of CKD does not increase the strength. They reported that incorporation CKD at low water-to-binder ratio to concrete mixtures does not show any negative effect. The mortars that were prepared with CKD had better absorption characteristics. However, above certain limits, the water absorption of the mortar increased which also lead to a decrease in the mortar strength.

Shoaib et al. [30] investigated the influence of CKD substitution on the mechanical properties of concrete. A total of 135 cubes and 135 cylinders were cast to study the effect of replacement of CKD on the mechanical behaviour of concrete. The concrete mixtures were prepared with 0%, 10%, 20%, 30%, and 40% CKD and tested after 1, 3, and 6 months. The mix proportion was cement - 1, sand - 1.9, gravel - 3.52, and w/c ratio – 0.5, the cement content used was 350 kg/m^3. It was reported that the ultimate compressive as well as tensile strength decreased with an increase in the quantity of CKD.

Shah and Wang [31] investigated utilization of CKD and Class F fly ash (FA) in concrete in the process of developing green concrete. The effects of mechanical, chemical and thermal activations on strength and other properties of CKD-FA binders were investigated. To achieve this, two combinations were made, one with CKD and FA ratio of 50:50 and another with 35:65. The hydration of CKD-FA binder was activated with addition of NaOH as chemical activation, exposing to curing temperatures of 38^0 C and 50^0 C as thermal activation and different grinding regimes were used as mechanical activation. Results

indicated that, when blend proportion and activation are properly applied, the binder made with CKD and fly ash will have satisfactory strength and performance, which provides potential applications for new cementitious product.

El-Sayed et al. [32] have investigated the effect of CKD on the compressive strength of cement paste and on the corrosion behavior of embedded reinforcement. They reported that substitution of up to 5% by weight of cement by CKD produced no adverse effect on the cement paste strength or on the reinfo rcement passivity.

Maslehuddin et al. [33] investigated CKD blended cement concrete specimens with 0%, 5%, 10%, and 15% CKD, replacing ASTM C 150 Type 1 and Type V cements. Mechanical properties and durability characteristics were assessed. Results indicated that compressive strength of concrete specimens decreased with the quantity of CKD and there was no significant difference in the compressive strength and drying shrinkage of 0 and 5% CKD cement concretes. The chloride permeability increased and the electrical resistivity decreased due to the incorporation of CKD. The performance of concrete with 5% CKD was almost similar to that of concrete without CKD. Therefore, it was suggested to limit the amount of CKD in concrete to 5% since the chloride permeability and electrical resistivity data indicated that the chances of reinforcement corrosion would increase with 10% and 15% CKD.

2.1.2 Electric Arc Furnace Dust (EAFD)

EAFD is generated as a by-product during the electric arc furnace steel making process. It is in the form of very fine powder forming major part of the smoke or fume from the furnace. The powder from the furnace is drawn through cooling pipes and collected in specially

designed bag filters. EAFD is a fine grained, high-density material containing high amounts of zinc and iron and substantial amounts of calcium, manganese, magnesium, lead and chromium. It is reported that about 15 to 20 kg of EAFD is generated for each ton of steel produced [34]. The annual steel production in Saudi Arabia is 471,000 tons [35]. Therefore, the annual production of EAFD in the Kingdom will approximately be 8,242 tons. During its production, the fine dust particles are released in atmosphere leading to a major pollution problem. As EAFD is considered hazardous by United States Environmental Protection Agency (US EPA), it must be stored in specialised landfill. However, due its high content of zinc, it can be used as secondary raw material for production of zinc or other products. The presence of zinc and pozzolanic materials will enhance the properties of concrete [34]. Hence, this material can be used for the production of medium to low strength concrete.
With replacement of EAFD, it is found that the setting time will retard, while the compressive and shearing strengths as well as resistance to abrasion will enhance [34].

Maslehuddin et al. [10] studied the mechanical properties and durability characteristics of ordinary Portland cement and blended cement (with silica fume and fly ash) concrete specimens with electric arc furnace dust (EAFD). Concrete specimens were prepared with and without EAFD. In the silica fume cement concrete, silica fume constituted 8% of the total cementitious material while fly ash cement concrete contained 30% fly ash. EAFD was added as 2% replacement of cement in the OPC concrete and 2% replacement of the total cementitious content in the blended cement concretes. Specimens were tested for compressive strength, drying shrinkage, initial and final setting time, slump retention, water absorption, chloride permeability, and reinforcement corrosion.

Results of that investigation indicated that the setting time and slump retention tended to increase with the addition of EAFD. However, there was a gain in strength with the addition of EAFD. Further, the water absorption and chloride permeability were found to decrease and there was an increase in the corrosion resistance of concrete with EAFD when compared to OPC and blended concretes.

Alexandre et al. [36] studied the waste behaviour of EAFD in Pozzolan-modified Portland cement paste (MP). To understand the residue effect and properties of cement paste in fresh and hardened states, setting time and heat of hydration were determined as well as mineralogical and micro structural characterization were evaluated. Results indicated that the EAFD retards the Portland cement's hydration reaction. At initial stages, the compressive strength was found to be less than control specimen but at advanced age significant gain was noted. The compressive strength with 5% EAFD was found to be similar to the reference MP cement paste at age of 28 days.

de Souza et al. [34] investigated the effect of EAFD on the mechanical and chemical performance of Portland cement concrete. They found that the compressive strength of concrete specimen increased with the addition of EAFD in the range of 10 to 20 wt. (%). Also, the tensile strength and setting time of specimens increased with the addition of EAFD and the chloride penetration decreased. The acetic acid leaching and water solubility test results show low movement of potentially toxic elements from EAFD-based concrete.

Xuefeng and Yuhong [37] assessed the physical properties and chemical composition of EAFD for its possible utilization in the cement production. Investigations were done and it

was reported that the quality of cement produced with EAFD meets the requirements for Chinese specifications. It was also reported that the use of EAFD in cement is more economical than the use of iron ore.

2.1.3 Limestone Powder (LSP)

Availability of LSP in Saudi Arabia is in abundance. LSP is obtained from the crushing of carbonate rocks which are primary sources of coarse aggregates in the Central and Eastern Provinces of the Kingdom. The usage of LSP in concrete has enormously increased over the last few years [38].

The addition of filler, like LSP, in concrete is considered as common practice in European countries, especially in France. Goals like technical, economic, and ecological fields can be achieved with the addition of LSP in cement. Technical benefits, like increase of early strength, control of bleeding in concrete with low cement content can be achieved [39]. From economical point of view, strength development with this type of cement is similar to that of Portland cement at low production and investment cost per ton of cement. Reduction of CO_2 and conservation of mineral resources are considered as ecological benefits [39]. LSP develops the hydration rate of cement compounds and increases the strength at early ages. LSP does not possess pozzolanic properties, but it reacts with the alumina phase of cement to form a calcium monocarboaluminate hydrate with no significant changes on the strength of blended cement. LSP has good ability of packing cement granular skeleton and a large dispersion of cement grains [39].

Liu and Yan [40] studied the effect of LSP on microstructure of concrete using mercury intrusion porosimetry (MIP), backscattering scanning electron (BSE), scanning electron microscopy (SEM) and X-ray diffraction (XRD) techniques. The mix proportion design was based on the strength grade of concrete with cement content of 110, 130, 160 and 200 kg/m^3 and to those mixes LSP of content 100 kg/m^3 was added as filler. They reported that the compressive strength of concrete containing 100 kg/m^3 limestone powder as addition can meet the strength requirement. Though LSP does not possess pozzolanic properties, its filling effect can make the paste matrix and the interfacial transition zone between matrix and aggregate denser, which improves the performance of concrete.

Heikal et al. [12] investigated the effect of substitution of LSP in pozzolanic cement. They reported that the initial and final setting times as well as the total porosity reduce with the addition of LSP. However, the content of free lime and combined water increased with limestone content. Formation of carboaluminate due to LSP fills the pores between cement particles, and that speeds up the setting time of cement. Addition of LSP results in reduction in the diffusion coefficient of chloride ions, and increases the heat of hydration and compressive strength.

Dhir et al. [41] investigated the performance of concrete produced by blending Portland cement and limestone, for mechanical and durability properties. They used 15, 25, 35 and 45% replacement of cement with LSP with a range of cement contents from 235 to 410 kg/m^3, and free water content of 185 l/m^3. They found that there were minor differences in the performance between Portland cement and 15% LSP blended cement concretes of the same cement and water-to-cement ratio. But, there was a decrease in the strength as the LSP

content increased. However, the flexural strength and modulus of elasticity decreased with an increase in the LSP content. Permeation and durability properties at equal w/c ratio enhanced up to 25% LSP and poorer performance thereafter. For the latter, minor effects were generally noted up to 15% LSP, but a gradual depletion in performance with increasing LSP in concrete thereafter.

Moon et al. [42] investigated the diffusion of chloride ions in concrete with and without LSP. In those mixes, cement was replaced with 0, 10, 20 and 30% of LSP content with a constant water-to-cementitious materials ratio of 0.45. They reported that the setting time of LSP was faster than that of control concrete. They observed that the compressive strength of all specimens decreased with increasing the content of LSP. Along with curing period, the trend of diffusion was found to increase. With the addition of 10 to 20% LSP, the diffusion coefficient was found to decrease.

Tahir and Khaled [43] investigated the effects of various proportions of LSP on fresh and hardened properties of concrete. The mixes were prepared by replacing fine aggregates partially. Several mixes were prepared with varying percentages of fine aggregates with LSP like 0, 5, 10, 15, 20, 25, and 30% with cement, water, fine aggregates, and coarse aggregates contents of 420, 210, 210, and 965 kg/m^3, respectively. They found that the slump decreased as the dust content increased. The compressive and flexural strength increased up to 10% and decreased gradually later. Absorption was found to increase with an increase in LSP after a dosage of LSP 15%. Drying shrinkage increased up to 10%, and decreased as replacement level of LSP increases more than 10%.

Tsivilis et al. [44] investigated the properties and behaviour of limestone cement concrete and mortar. The limestone cements were produced by grinding clinker, limestone and gypsum with varying percentages of limestone and clinker. The mixes were prepared with clinker replacing 0, 10, 15, 20, and 35% with limestone, and 0.5 water-to-cement ratio. For durability evaluation, the specimens were exposed to 3% NaCl solution. The compressive strength and workability of LSP concrete were similar to that of control concrete. The 20% LSP was found to be optimum for protection against reinforced corrosion. Sorptivity and chloride permeability were found to be similar to control concrete. The freezing thawing of LSP concrete was less when compared to control concrete. With the addition of limestone, the carbonation depth and total porosity of mortar decreases.

2.1.4 Oil Ash (OA)

OA is a by-product of the burning oil to produce energy in power generation plants. It consists of inorganic substances, such as SiO_2, Al_2O_3, Fe_2O_3, with 70~80% of unburned carbon and heavy metals, like vanadium and nickel, that are present in the crude petroleum at the initial stages.

Saudi Arabia has the world's largest proven reserves of oil, and it is readily available and economically feasible for generation of power. Saudi Arabia's Water and Electricity Ministry estimates that the country will require at least 30 gigawatts of additional power generation capacity by 2023-25 [http://www.eoearth.org/article/Energy_profile_of_Saudi_Arabia]. Saudi Arabia is investing heavily in increasing the power and drinking water capacity. Shuaibah is the first power and water project in Saudi Arabia, and the first of a total of four

planned major projects. The goal of these projects is to increase the power plant capacity by 4,500 MW and to provide an additional 2.2 million cubic meters of drinking water daily [17].

The consumption of oil reaches 1 million barrel per day (bpd) of crude during summer months and it is likely to increase to 8 million bpd by 2030 if there are no improvements in energy efficiency [45]. Hence, research has to be done to explore the utilization of OA generated.

Abdullah [17] has recently investigated the stabilization of two eastern Saudi soils, namely non-plastic marl and sand utilizing CKD and OA. OA content of 0, 5, 10, and 15% was used with 0 and 5% cement content to stabilize non-plastic marl and sand. Several tests were performed to assess the engineering properties of soils with and without stabilizer. It was reported that OA was found to be a suitable chemical addition to treat non-plastic marl soil. OA content of 5% plus 5% cement was found to be adequate for the effective stabilization of non-plastic marl. It met the strength and durability requirements.

Most of the studies have addressed fly ash generated from burning coals. Literature on reuse and/or recycling of oil ash generated from combustion of heavy fuel oil is very scarce because of the limited use of this oil for power generation. Therefore, specific research programs should be initiated to identify possible uses for this type of oil ash.

2.1.5 Natural Pozzolan (NP)

NP is historically the oldest material that has been used for construction purposes after blending with lime. NP is either used as admixture in concrete or for the production of Portland-pozzolan cements. NP is most widely used in regions where there is lack of other

pozzolans like silica fume, fly ash and slag, for economic production of concrete due to its beneficial properties, like low heat of hydration, low permeability, high sulphate resistance, and low alkali silica activity. It's another importance is regarding the sustainability of cement and concrete industry from energy efficiency and environmental aspect [18].

In the Kingdom of Saudi Arabia, locally available pozzolan has potential for use. In the Eastern and Western regions, natural pozzolanic material is used to help in preventing the deterioration of concrete. NP is available in the basalt plateaus (Harrat) spread within the "Edge of Arabian Shield". These plateaus are spread across an area of 90,000 km^2 in the east of the escarpment onwards to the coast of the Red Sea. The natural pozzolan of various locations like Yanbu, Rabaig, and Tabuk, generally showed similar chemical and physical properties. NP fineness and source did not affect compressive strength. Hot curing is very beneficial for strength development and for improvement in chloride permeability [46]. Hence, utilization of locally available NP in hot weather regions of the Kingdom of Saudi Arabia is very favourable.

Najimi et al. [47] investigated the effect of natural pozzolan on Portland cement. Concrete mixtures with 25% replacement of cement with natural pozzolan were studied. The blended and control mixes were tested for mechanical and durability properties. Compressive strength of specimens after 180 days was slightly decreased (i.e., less than 5%). The specimens with pozzolan had slightly enhanced the modulus of elasticity and decreased chloride ion permeability, but did not perform well in freeze and thaw and sulphate expansion tests when compared to control specimen. Due to the lower content of amorphous silica in natural

pozzolan, it was also found that the hydration rate was slow. Therefore, the best properties were obtained after 90 and 180 days of curing.

Fajardo et al. [48] investigated the corrosion of steel in reinforced concrete which is most common durability problem. They prepared reinforced mortar specimens by substituting Portland cement partially with a Mexican natural pozzolan. The level of substitution of natural pozzolan was 0, 10, and 20% by mas of Portland cement. The mortar specimens were exposed to chloride penetration and were investigated for compressive strength, corrosion potential, polarization resistance, electrical resistivity, and chloride content. It was reported that the partial replacement of Portland cement with natural pozzolan has beneficial effect on corrosion as reflected by higher resistivity of mortars, and reduction in corrosion rate. The compressive strength was found to decrease with an increase in the pozzolan content, which is acceptable taking in account the reduction of the chloride ingress.

Pekmezci and Akyuz [49] investigated the optimum usage of natural pozzolan for maximum compressive strength of concrete. Fifteen concrete mixtures were prepared in three series with control mixes having 300, 350, and 400 kg cement content. Each of these control mixes were modified to have combination of 250, 300, and 400 kg cement content and 40, 50, 75, and 100 kg pozzolan addition for 1 m^3 concrete. They used Bolomey and Feret strength equations on 28-day old concrete to find the efficiency of pozzolan. It was reported that the optimum pozzolan/cement ratio to obtain the maximum strength is approximately 0.28 and efficiency decreases with an increase in the pozzolan/cement ratio. When equal amount of pozzolan is used, concrete with highest cement content has the highest efficiency of pozzolan.

2.1.6 Medium to Low Strength Concrete

Some of these low strength concrete are referred to as controlled low strength materials (CLSM) in the literature. CLSM is a flowable mixture that can be used as a backfill material in place of compacted soils [50]. Flowable fill requires no tamping or compaction to achieve its strength and typically has a load-carrying capacity much higher than that of compacted soils, but it can still be excavated easily [51].

Although CLSM generally costs more per cubic meter than most soil or granular backfill materials, it has many advantages, such as reduced labour, self-levelling capability and versatility, leading to overall low cost [52]. CLSM is basically a mixture of cement, a by-product material (usually fly ash), fine aggregate and water that can be used as a backfill material in place of compacted soils with its self-leveling property [51]. Recycling of waste materials for use in CLSM is also helpful in the conservation of environment. Fly ash, Class F or Class C could be utilized in CLSM fills with relatively low proportions of cement to activate pozzolanic reactions. Besides FA, various by-products, such as acid mine drainage (AMD) sludge, quicklime (QL), have also been used [53].

Several researchers have evaluated the use of various industrial by-products in the production of CLSM. Some of these waste materials are FA, bottom ash, blast furnace slag, foundry sand, pulverized rubber tires, CKD, and flue gas desulfurization (FGD) materials [54-57]. Turkel [50] investigated the properties of a mixture of high volume fly ash (FA), crushed LSP (filler) and a low percentage of pozzolana cement in different compositions. The amount of pozzolana cement was kept constant for all mixes as 5% of the fly ash weight. The amount of

mixing water was chosen in order to provide optimum pumpability by determining the spreading ratio of CLSM mixtures using the flow table method. The shear strength properties of CLSM mixtures were investigated by a series of laboratory tests. The test results indicated that CLSM mixtures have superior shear strength properties compared to compacted soils. Shear strength, cohesion intercept and angle of shearing resistance values of CLSM mixtures exceeded conventional soil materials' similar properties at seven days. These parameters proved that CLSM mixtures are suitable for backfill applications.

Nataraja and Naland [52] investigated the performance of industrial by-products in CLSMs. Three industrial byproducts, namely FA, rice husk ash (RHA) and quarry dust (QD), were used as constituent materials in CLSM. Mixture proportions were developed for CLSM containing these industrial by-products and were tested in the laboratory for various properties, such as flowability, unconfined compressive strength (UCS), stress–strain behaviour, density, water absorption and volume changes. Comparison between the two pozzolanic materials, namely FA and RHA, for their potential to produce an effective CLSM was made. Results indicated that the by-product materials, such as FA, RHA and QD, can be successfully used in CLSM.

The low-strength property of CKD can be advantageous when used in CLSM because many applications require future excavatability [58]. The flowability and setting times within 24 hours could be achieved with most mixtures. CKD can be beneficially added to produce a very low-strength material due to the smaller amount of lime and silica present in CKD, which offers comparable strengths to soils used for conventional fills and many other low-strength applications.

CKD can also be utilized in the flowable fill mixtures. The strength of the mixtures varies quite significantly because of the considerable difference in the amount of water added to reach the required slump. According to Taha et al. [59], the quantities of materials used in each mixture also influence the strength and slump values. Foundry sand is becoming a more viable product for use in CLSM because of its lower cost and increasing availability. By-product foundry sand used in CLSM provides better properties compared to that of CLSM containing crushed limestone sand [60]. Spent casting sand can also be used as a constituent in CLSM. The uniform size of the spent casting sand can provide good flowability, and its lower cost enhances its economic advantage as a constituent in CLSM [61].

According to Nmai et al. [62], CLSM can be classified as either regular or low density mixtures that have a density of 800 kg/m^3 or more are generally classified as regular CLSM (Reg-CLSM), and those with a density less than that are classified as low-density CLSM (LD-CLSM). Regular CLSM consists of cement, FA, fine aggregates, water and/or air-entraining admixtures, whereas LD-CLSM consists of a foaming agent in addition to cement, FA and water.

In a study conducted by Katz and Kovler [63], industrial by-products were used for the production of controlled low-strength material (CLSM). In that study, CKD, asphalt dust, coal fly ash, coal bottom ash, and quarry waste were tested for the possibility of producing CLSM with large proportions of those wastes. The results showed that in most cases, CLSM with good properties could be made with significant amounts of dust (25-50%w), especially when the dust has some cementing or pozzolanic potential as do FA and CKD.

Taha et al. [59] evaluated the potential use of CKD, incinerator ash, and copper slag in a CLSM. Mixtures were designed to produce a CLSM with a low compressive strength (less than 1,034 kPa), that can be excavated without using any mechanical equipment. Slump, unit weight and unconfined compressive strength were evaluated on various mixtures. Cubical and cylindrical specimens were prepared and cured at room temperature and in sealed plastic bags. Results indicated that with a good mix design, it is possible to produce a CLSM with good mechanical properties to meet the design requirements. Mixing these materials with cement and sand produced better results than using them alone due to their low pozzolanic activity. Curing method and period can have considerable effects on the strength of a CLSM.

Lachemi et al. [64] evaluated the feasibility of incorporating CKD in the development of CLSM. A preliminary study (Phase I) was conducted (based on fresh and strength properties) to understand the behaviour of 12 selected CLSM mixtures where CKD and cement content varied from 4% to 45% and from 2% to 4% of total mass, respectively. Subsequently, four best CLSM mixtures were selected for a detailed study (Phase II), which investigated fresh and hardened properties, addressed durability issues, and made recommendations for suitable mix designs for field applications. The research suggests that CLSM with acceptable properties can be developed using moderate volumes of CKD (up to 15% by mass). A combination of 2% cement and 10% CKD or 15% CKD with no cement can provide a mix that satisfies the requirements of a CLSM.

Siddique and Kadri [65] investigated the use of metakaolin (i.e., a refined kaolin clay that is fired (calcined) under carefully controlled conditions to create an amorphous aluminosilicate that is reactive in concrete). Like other pozzolans (fly ash and silica fume are two common

pozzolans), metakaolin reacts with the calcium hydroxide (lime) that is produced during cement hydration. The authors replaced cement with 5%, 10% and 15% metakaolin and foundry sand replacing 20% fine aggregate. The control concrete was designed with cement content of 450 kg/m^3 and w/c of 0.45. Results indicated that there was a decrease in the initial surface absorption with an increase in metakaolin from 5 to 15% and decrease in sorptivity till 10% metakaolin replacement. Compressive strength decreased with an increase in the metakaolin and with addition of foundry sand.

Ergun [66] investigated the mechanical properties of concrete using diatomite and waste marble powder as replacement for cement. Diatomite is a pozzolanic material that contains amorphous silica, cristabolite and negligible amounts of residual minerals. Waste marble powder (WMP) is an industrial by-product during sawing, shaping, and polishing of marble and causes a serious environmental problem. To assess concrete specimens, cement was replaced with following percentages of 5%, 7.5% and 10% diatomite and WMP replacement for Series-I and Series-II and 5% and 10% diatomite and WMP together replacement for Series-III were used. Results indicated that specimens containing 10% diatomite, 5% WPM and 5% WPM +10% diatomite replacement by weight for cement had the best compressive and flexural strength.

Srinivasan et al. [67] investigated the utilization of hypo sludge; a waste by-product from paper industry, for strength of concrete and optimum percentage by replacing 10, 20, 30, 40, 50, 60 and 70% of hypo sludge with cement. It was reported that compressive strength increased up to 30% replacement and then reduced as the replacement increases. Split tensile strength was reduced as replacement increases.

Pacheco-Torgal and Jalali [68] investigated reusing of ceramic wastes with partial replacement of cement by ceramic powder and the replacement of traditional aggregates by ceramic sand and ceramic aggregates. The 20% cement was replaced with ceramic powder of particle size less than 75 μm. It was reported that there was a minor strength loss due to the addition of ceramic waste powder but there was good increase in durability due to its pozzolanic properties.

Pazhani and Jeyaraj [69] investigated the utilization of industrial waste copper slag and ground granulated blast furnace slag (GGBFS); a by-product of copper refinery and manufacture of iron in steel industry, respectively. Durability of concrete was assessed by replacing some percentage of fine aggregate with copper slag and some percentage of cement with GGBFS. It was reported that with 30% replacement of cement with GGBFS, water absorption decreased by 4.58%, and water absorption for 100% replacement of fine aggregate with copper slag decreased by 33.59%. Due to GGBFS, chloride ion penetrability decreased by 29.90% and for copper slag 77.32% and pH decreased by 0.39% for GGBFS and for copper slag by 3.04% when compared to control specimens.

Kharaazi and Eshmaiel [70] investigated over 100 waste products for their pozzolanic properties and proper replacement for cement for production of cheap concrete. It was reported that travertine rock powder (TRP) (i.e., a limestone that forms by evaporative precipitation, often in a cave, to produce formations such as stalactites, stalagmites and flowstone), paper factory sludge (PFK) (i.e., a by-product of the de-inking and re-pulping of paper at paper mills), ferro-silica powder (FSP) (i.e., widely used in casting, melting and related metallurgy industry and for production of semiconductors), heated brick kiln wastes

(LHB) (i.e., waste recovered at brick kilns), heated Kaolin of tile factory (MK)(i.e., a refined kaolin clay that is fired (calcined) under carefully controlled conditions to create an amorphous aluminosilicate at tile factory), cast iron mills slag (SIS) (i.e., residue generated during generation of iron from iron ore), and weld factory slag (WFS) (i.e., a residue formation during welding at factories), show excellent pozzolanic properties and can be used as cement replacement materials in concrete. The best composition for replacement of cement was given as 75 percent by weight MK and 25 percent by weight FSP together.

Mesci et al. [71] investigated the utilization of the chromite industry waste and red mud for production of concrete. Chromite ore processing residue is the solid waste remaining after the aqueous extraction of oxidized chromite ore that is combined with soda ash and kiln roasted at high temperature. Chromium is a trace of metallic element that is found in the earth's crust in concentrated ore. These two products were incorporated into concrete by replacing 5, 10, and 15% cement. Compressive strength of concrete with 5% replacement of both chromite industry waste and red mud was similar to that of the control mix and higher percentage of the chromite industry waste and the red mud (15%) replacement yielded lower strength values.

Naceri and Hamina [72] investigated the properties of cement mortar with 0, 5, 10, 15 and 20% replacement of cement clinker with waste brick (i.e., generated during manufacture of bricks). The grinding time of the cement containing waste brick was lower than the control cement because of low abrasive property and specific gravity of the waste brick, setting time of cement paste decreased due to high water absorption and specific weight. The substitution of cement by 10% of waste brick increased the mechanical strengths of mortar.

Zainab and Al-Hashmi [73] carried out 109 experiments and 586 tests to study the feasibility of reusing waste iron in concrete. The sand was replaced with waste iron in proportions of 10%, 15% and 20%. It was reported that the slump decreased with an increase in the quantity of waste iron. Fresh and dry densities, compression and flexural strengths increased with an increase in the waste iron than the plain concrete mixes.

While waste materials have been utilized to produce CLSM and other concretes in other parts of the world, research is yet to be carried out in the Kingdom of Saudi Arabia on the use of local waste materials, particularly CKD, EAFD, LSP, OA, and NP for the production of medium to low strength concrete. Such materials could be utilized for blinding concrete, building pavements, filling electrical conduits, as back fill material, preparing masonry blocks, etc. Such usage would result in a less consumption of cement and, hence, lower the production of greenhouse gas emissions.

CHAPTER 3

EXPERIMENTAL PROGRAM

This chapter addresses the materials characterization and experimental methods utilized in this study. This research focuses on the production of medium to low strength concrete using locally available waste materials, (i.e., CKD, LSP, EAFD, OA, and NP). To achieve the proposed work, the following phases were followed. First phase was to procure the concrete ingredients and waste products. In the second phase, preparation of specimens was carried out and in the third phase, testing of specimens was done to ascertain the mechanical, thermal and electrical properties. In this chapter, all these three phases are discussed thoroughly.

3.1 Materials

3.1.1 Cement

Ordinary Portland cement conforming to ASTM C 150 Type I with a specific gravity of 3.15 was used in all the concrete mixtures. Sufficient amount of cement was procured and stockpiled safely to prevent hardening of cement. The chemical composition of this cement was done by research institute, KFUPM as shown in Table 2.1.

Table 3-1: Chemical Composition of Cement.

Constituent	**Weight %**
CaO	64.35
SiO_2	22.0
Al_2O_3	5.64
Fe_2O_3	3.80
K_2O	0.36
MgO	2.11
Na_2O	0.19
Equivalent alkalis ($Na_2O + 0.658K_2O$)	0.33
SO_3	2.10
Loss on ignition	0.7
C_3S	55
C_2S	19
C_3A	10
C_4AF	7

3.1.2 Fine Aggregates

Dune sand with water absorption of 0.6% and specific gravity of 2.56 was used as the fine aggregate.

3.1.3 Coarse Aggregates

Four aggregate sizes of 12.5 mm (½ inch), 9.5 mm (3/8 inch), 4.75 mm (3/16 inch), and 2.36 mm (3/32 inch) crushed limestone from Riyadh Road were used in all the concrete mixtures. Absorption and specific gravity of the coarse aggregates were 1.1% and 2.6, respectively.

3.1.4 Superplasticizer

Varying dosage of superplasticizer (SP 430) was used to obtain a slump of 100 ± 25 mm for all the mixes.

3.2 Waste Products

3.2.1 Cement Kiln Dust (CKD)

CKD was obtained from the Saudi Arabian Cement Company, Jeddah, Western Saudi Arabia. The chemical composition of CKD was done by research institute, KFUPM as shown in Table 3.2.

Table 3-2: Chemical Composition of CKD.

Constituent	**Weight %**
CaO	49.3
SiO_2	17.1
Al_2O_3	4.24
Fe_2O_3	2.89
K_2O	2.18
MgO	1.14
Na_2O	3.84
P_2O_5	0.12
Equivalent alkalis ($Na_2O + 0.658K_2O$)	5.27
SO_3	3.56
Chloride	6.90
Loss on ignition	15.8
BaO (μg/g (ppm))	78.2
Cr_2O_3	0.011
CuO	0.029
NiO	0.012
TiO_2	0.34
V_2O_5	0.013
ZnO (μg/g (ppm))	65.8
ZrO_2	0.011

3.2.2 Electric Arc Furnace Dust (EAFD)

EAFD was procured from the Saudi Iron and Steel Company (HADEED). The chemical composition of EAFD was done by research institute, KFUPM as shown in Table 3.3.

Table 3-3: Chemical Composition of EAFD.

Constituent	**Weight %**
Aluminium	0.7
Calcium	9.39
Cadmium	0.0004
Copper	0.06
Iron	33.6
Potassium	1.70
Magnesium	2.3
Manganese	1.8
Sodium	2.6
Nickel	0.01
Lead	1.31
Phosphorous	0.13
Silicon	2.38
Tin	0.03
Sulphur	0.57
Titanium	0.09
Zinc	10

3.2.3 Limestone Powder (LSP)

LSP was obtained from a quarry in Abu Hadriyah, Eastern Province, Saudi Arabia. The chemical composition of LSP was done by research institute, KFUPM as shown in Table 3.4.

Table 3-4: Chemical Composition of LSP.

Constituent	**Weight %**
SiO_2	11.79
CaO	45.7
Al_2O_3	2.17
Fe_2O_3	0.68
MgO	1.80
K_2O	0.84
Na_2O	1.72
$Na_2O+(0.658K_2O)$	2.27
Loss on Ignition	35.10
Moisture	0.20

3.2.4 Oil Ash (OA)

OA was procured from the Saudi Electricity Company power plant in Shayba, Saudi Arabia. The chemical composition of OA was done by research institute, KFUPM as shown in Table 3.5.

Table 3-5: Chemical Composition of OA.

Constituents	**Weight %**
SiO_2	1.65
CaO	0.45
Al_2O_3	< 0.10
Fe_2O_3	0.47
MgO	0.48
K_2O	0.03
Na_2O	0.53
V_2O_5	2.65
Sulfur	9.6
$Na_2O + (0.658K_2O)$, %	0.55
Loss on ignition	60.6
Moisture %	5.9

3.2.5 Natural Pozzolan (NP)

NP was obtained from volcanic rocks in Western Province of Saudi Arabia. The chemical composition of NP was done by research institute, KFUPM as shown in Table 3.6.

Table 3-6: Chemical Composition of NP.

Constituent	**Weight %**
SiO_2	42.13
Al_2O_3	15.33
Fe_2O_3	12.21
MgO	8.5
K_2O	0.84
Na_2O	2.99

3.3 Mix Proportions

To achieve the objectives of project of this test program, 23 concrete mixtures resulting in 690 specimens were cast for laboratory testing. The details of the various proportions of concrete mixes were divided into the following categories:

(i) Medium strength with a cementitious materials content of 370 kg/m^3, water to cementitious materials ratio of 0.45 and low dosage of waste materials.

(ii) Low strength with a cementitious materials content of 300 kg/m^3, water to cementitious materials ratio of 0.55 and high dosage of waste materials.

3.3.1 Medium Strength Concrete (MSC) with CKD

MSC specimens were prepared with CKD used as low partial replacements of cement (5, 10 and 20%). They were prepared with a cementitious materials content of 370 kg/m^3 and a water to cementitious materials ratio of 0.45.

3.3.2 Low Strength Concrete (LSC) with CKD

LSC specimens were prepared with CKD used as high partial replacements (50 and 60%) of cement. They were prepared with a cementitious materials content of 300 kg/m^3 and a water to cementitious materials ratio of 0.55.

3.3.3 Medium Strength Concrete with EAFD

MSC specimens were prepared with EAFD used as low partial replacements of cement (5, 10, and 15%). They were prepared with a cementitious materials content of 370 kg/m^3 and a water to cementitious materials ratio of 0.45.

3.3.4 Low Strength Concrete with EAFD

LSC specimens were prepared with EAFD used as high partial replacements (50 and 60%) of cement. They were prepared with a cementitious materials content of 300 kg/m^3 and a water to cementitious materials ratio of 0.55.

3.3.5 Medium Strength Concrete with LSP

MSC specimens were prepared with LSP used as low partial replacements of cement (5, 10, and 15%). They were prepared with a cementitious materials content of 370 kg/m^3 and a water to cementitious materials ratio of 0.45.

3.3.6 Low Strength Concrete with LSP

LSC specimens were prepared with LSP used as high partial replacements (50 and 60%) of cement. They were prepared with a cementitious materials content of 300 kg/m^3 and a water to cementitious materials ratio of 0.55.

3.3.7 Medium Strength Concrete with OA

MSC specimens were prepared with OA used as low partial replacements of cement (5 and 10%). They were prepared with a cementitious materials content of 370 kg/m^3 and a water to cementitious materials ratio of 0.45.

3.3.8 Low Strength Concrete with OA

LSC specimens were prepared with OA used as high partial replacements (50 and 60%) of cement. They were prepared with a cementitious materials content of 300 kg/m^3 and a water to cementitious materials ratio of 0.55.

3.3.9 Medium Strength Concrete with NP

MSC specimens were prepared with local NP used as low partial replacements of cement (0, 10, 15 and 20%). They were prepared with a cementitious materials content of 370 kg/m^3 and a water to cementitious materials ratio of 0.45.

3.3.10 Low Strength Concrete with NP

LSC specimens were prepared with NP used as high partial replacements (50 and 60%) of cement. They were prepared with a cementitious materials content of 300 kg/m^3 and a water to cementitious materials ratio of 0.55.

Weights of materials used in all the concrete mixes are provided in Table 3.7.

Table 3-7: Weights of Ingredients in the Mixtures Investigated.

WASTE PRODUCT	MIX CODE	PERCENTAGE REPLACEMENT (%)	CEMENTETIOUS MATERIALS (kg/m³)	CEMENT I (kg/m³)	WASTE PRODUCT (kg/m³)	W/C RATIO	COARSE AGGREGATE (kg/m³)				FINE AGGREGATE (kg/m³)
							1/2 in	3/8 in	#4	#8	Dune sand
	PLAIN	0	370	370	0	0.45	516.35	172.12	344.23	114.74	703.27
CKD	CKD-5	5	370	351.5	18.5	0.45	515.80	171.93	343.87	114.62	702.53
	CKD-10	10	370	333	37	0.45	515.26	171.75	343.50	114.50	701.78
	CKD-20	20	370	296	74	0.45	514.16	171.39	342.78	114.26	700.29
	CKD-50	50	300	150	150	0.55	529.03	176.34	352.68	117.56	720.54
	CKD-60	60	300	120	180	0.55	528.14	176.05	352.09	117.36	719.33
BHD	BHD-5	5	370	351.5	18.5	0.45	515.45	171.82	343.64	114.55	702.05
	BHD-10	10	370	333	37	0.45	514.56	171.52	343.04	114.35	700.83
	BHD-15	15	370	314.5	55.5	0.45	513.66	171.22	342.44	114.15	699.61
	BHD-50	50	300	150	150	0.55	526.19	175.40	350.79	116.93	716.68
	BHD-60	60	300	120	180	0.55	524.74	174.91	349.83	116.61	714.7
LSP	LSP-5	5	370	351.5	18.5	0.45	511.01	170.34	340.68	113.56	696.01
	LSP-10	10	370	333	37	0.45	505.68	168.56	337.12	112.37	688.74
	LSP-15	15	370	314.5	55.5	0.45	500.34	166.78	333.56	111.19	681.47
	LSP-50	50	300	150	150	0.55	490.20	163.40	326.80	108.93	667.66
	LSP-60	60	300	120	180	0.55	481.55	160.52	321.03	107.01	655.87
OA	OA-5	5	370	351.5	18.5	0.45	498.35	166.12	332.23	110.74	678.76
	OA-10	10	370	333	37	0.45	480.35	160.12	320.23	106.74	654.24
NP	NP-10	10	370	333	37	0.45	528.84	176.28	352.56	117.52	720.29
	NP-15	15	370	314.5	55.5	0.45	515.71	171.90	343.81	114.60	702.41
	NP-20	20	370	296	74	0.45	515.50	171.83	343.67	114.56	702.12
	NP-50	50	300	150	150	0.55	531.74	177.25	354.49	118.16	724.23
	NP-60	60	300	120	180	0.55	531.40	177.13	354.26	118.09	723.76

3.4 Preparation and Curing of Specimens

Concrete specimens were prepared and cured to carry out various tests planned in this investigation. Batching of each mix was proportioned by weight. Aggregates were initially

sieved to obtain the required sizes. The concrete constituents were thoroughly mixed in an electrically operated mixer of 1.7 m^3 capacity till a uniform consistency was obtained. After mixing, the slump was measured then concrete was poured in the moulds. The moulds were then vibrated until complete consolidation was achieved after a thin film of mortar appeared on the concrete surface. After casting, the specimens were covered with plastic sheet for 24 hours in the laboratory environment (22 $\pm$ 3^0 C) to minimize loss of mix water. After 24 hours, the specimens were demoulded and placed in a curing tank till the time of test. Table 3.8 shows the type and number of specimens used in this investigation.

Table 3-8: Type and Number of Specimens Prepared and Tested.

Test	**Specimen Type**	**Dimensions (mm)**	**Test Standard**	**Specimens Tested**
Compression	Cubes	100	ASTM C 39	345
Water Absorption	Cylinders	75 X 150	ASTM C 642	69
Thermal Conductivity	Cubes	100	ASTM C 201	69
Electrical Resistivity	Cubes	75 X 150	Two-probe Wenner method	69
Corrosion Potentials	Cylinders with reinforcement	75 X 150	ASTM C 876	69
Corrosion Current Density	Cylinders with reinforcement	75 X 150	LPRM	
Setting Time	*	*	ASTM C 191	
Leachability of Heavy Metals	Cubes	100	TCLP	Specimens used from another test
Drying Shrinkage	Prisms	40 x 40 x 160	ASTM C 157	69
Total Number of Specimens				690

* Plastic concrete was used

3.5 Testing of Specimens

3.5.1 Setting Time

The term setting is used to describe the stiffening of the cement paste. Initial set indicates that the paste is beginning to stiffen considerably and can no longer be moulded. Final set indicates that the cement has hardened to the point at which it can sustain marginal load. Setting time was determined using the Vicat apparatus according to ASTM C 191 [80]. Initial setting time is the time from the instant at which water is added to the cement to the instant at which the Vicat's initial set needle penetrates 25 mm into the cement paste. Final setting time corresponds to the time at which the needle does not sink visibly into the cement paste. The setup used to determine the initial and final setting time is shown in Figure 3.9.

Figure 3-1: Vicat Apparatus.

3.5.2 Compressive Strength

Compressive strength of a material is that value of the uniaxial compressive stress reached when the material fails completely. Strength test results of cubes can be used for quality control purposes. Compressive strength was calculated from the failure load divided by the cross-sectional area resisting the load and reported in megapascals (MPa). Compressive strength was determined on 100 mm cube specimens according to ASTM C 39 [74] using a digital compression testing machine (MATEST) after 3, 7, 14, 28 and 90 days of water curing. Three specimens were tested at each age and the average values are reported. Figure 3.1 shows the compression testing machine (MATEST).

Figure 3-2: Compression Testing Machine.

3.5.3 Water Absorption

After hydration of cement paste, some voids will be left behind after the hydration process which affects the strength and durability of concrete. With the presence of air voids in concrete, it is vulnerable to penetration and attack by aggressive species (i.e., sulfates, chlorides, CO_2 etc.). Good quality concrete is characterized by having minimal voids left by excess water and, therefore, water absorption test is adopted for assessing the quality of concrete in terms of density, durability and imperviousness.

Water absorption was determined using 75 mm diameter and 150 mm high cylindrical specimens according to ASTM C 642 [75] after 28 days of water curing. First, the specimens were dried in oven for 24 hours at a temperature of 110^0C and then their weights were recorded. They were then soaked in water for 48 hours and their saturated surface dry weights were taken. Water absorption was calculated utilizing the following formula.

Weight of saturated surface dried sample = A

Weight of oven dried sample = B

$$\text{Water Absorption} = \frac{A - B}{B} \times 100\%$$

Three specimens were tested and the average values were reported. Equipment used for determining the water absorption is shown in Figure 3.2.

a) Specimens dried in oven

(b) Specimens immersed in water

(c) Cylindrical Specimens

Figure 3-3: Equipment Used to Determine Water Absorption .

3.5.4 Thermal Conductivity

The heat of hydration of concrete or occurrence of temperature gradient may lead to micro-cracks in a concrete structure. To predict cracking response and to cope with it, a systematic way to do analysis on the heat conduction is needed. The thermal conductivity was evaluated on 100 mm cube specimens according to ASTM C 201 [76] using thermal conductivity equipment from C-Therm after 28 days of water curing. The sensor was placed on top of the cube sample and then the value was recorded. The interface between the concrete surface and sensor was moistened with water. Three specimens were tested and the average values were reported. Apparatus is shown in Figure 3.3.

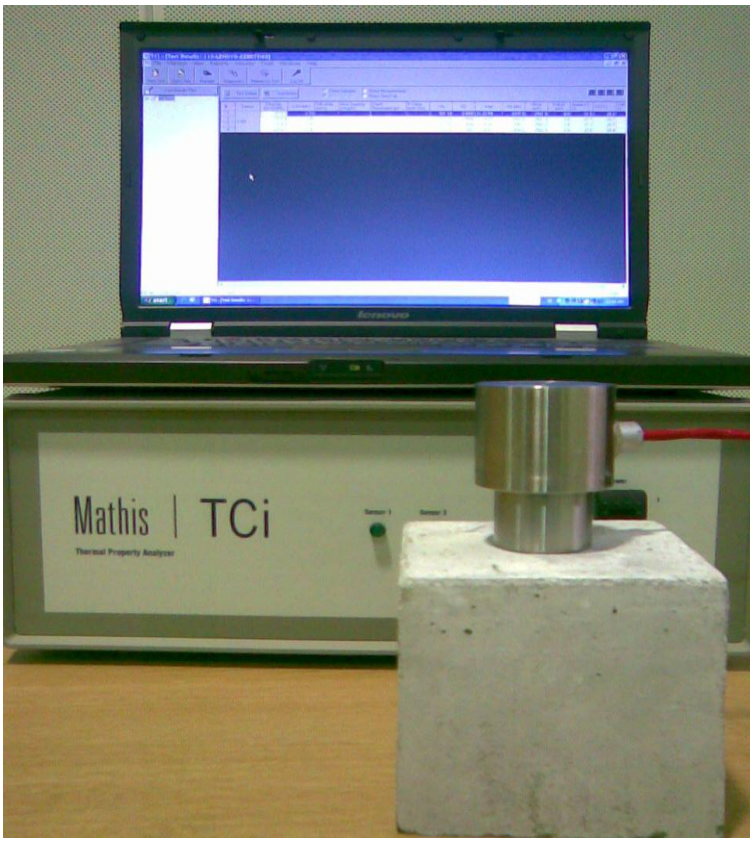

Figure 3-4: Thermal Conductivity Apparatus.

3.5.5 Electrical Resistivity

Electrical resistivity of concrete determines the probability of degree of reinforcement corrosion. Concrete resistivity is a geometry-independent "isotropic" material property that describes the electrical resistance, which is the ratio between applied voltage and resulting current in a unit cell. The dimension of resistivity is resistance multiplied by length and its unit usually is in Ω-m. In concrete, the current is carried by ions dissolved in the pore liquid. More pore water (wet concrete) as well as more and wider pores (high water to cement ratio) cause a lower resistivity while high resistivity indicates good and dense concrete.

The electrical resistivity was determined on 75 mm diameter and 150 mm high cylindrical concrete specimens after 28 days of water curing at different moisture contents according to two-probe Wenner method [77]. For this purpose, Hipotronics, digital ground resistance tester, was used. This is a rugged, easy to use tester designed especially for measuring low to high electrical resistance. This is state-of-the-art equipment capable of measuring up to 20 kOhms resistance with a resolution of 1 mOhm. The specimens were initially saturated in the water and the electrical resistivity was measured. It was then allowed to dry in the laboratory environment and the electrical resistivity was measured. The classification of electrical resistivity with regard to the risk of reinforcement corrosion is summarized in Table 3.9 [33]. Three specimens were tested and the average values were reported. The apparatus is shown in Figure 3.4.

Table 3-9: Concrete Resistivity and Risk of Reinforcement Corrosion [33].

Concrete resistivity (kΩ-cm)	Risk of reinforcement corrosion
<10	High
10–50	Moderate
50–100	Low
>100	Negligible

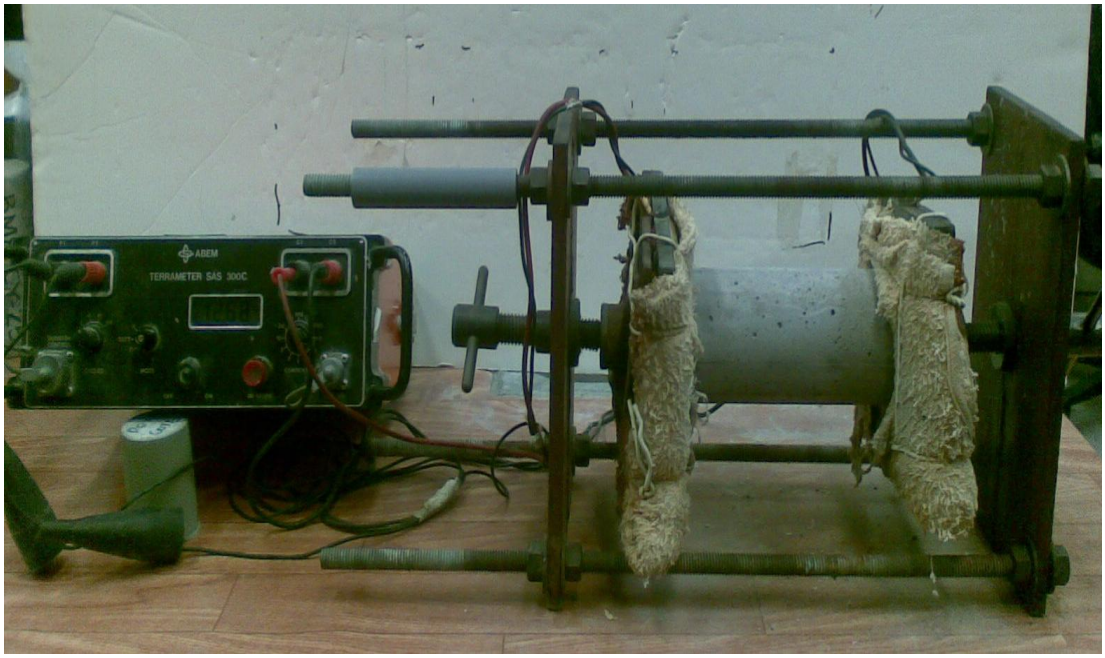

Figure 3-5: Electrical Resistivity Apparatus.

3.5.6 Reinforcement Corrosion

Reinforcement corrosion was evaluated by measuring corrosion potentials and corrosion current density. The concrete specimens were exposed to 5% NaCl and potentials were measured after every two weeks while corrosion current density was measured every month.

3.5.6.1 Corrosion Potential Measurement

Corrosion potentials were measured to evaluate the probability of reinforcement corrosion. The corrosion status is related to the measured corrosion potential value. For this purpose, reinforcement corrosion was monitored by exposing reinforced concrete specimens, 75 mm in diameter and 150 mm high with a centrally placed 12 mm diameter steel bar, as shown in Figure 3.5, to 5% sodium chloride solution after 28 days of curing and corrosion potentials were measured according to ASTM C 876 [78]. Saturated calomel electrode (SCE) was used as the reference electrode. If the potential reaches a threshold value -270 mV, then there is a 90% probability for initiation of reinforcement corrosion. Three specimens were tested and the average values were reported. The potential measurement setup is depicted in Figure 3.6. Table 3.10 indicates the possibility of corrosion in reinforcement as reported in ASTM C 876.

Table 3-10: Probability of Occurrence of Reinforcement Corrosion (ASTM C 876).

Open circuit potential (OCP) values		**Corrosion condition**
(mV vs. SCE)	**(mV vs. CSE)**	
< -426	< -500	Severe corrosion
< -276	< -350	High (<90% risk of corrosion)
> -125 but < -276	> -200 but < -350	Intermediate corrosion risk
> -125	< -200	Low(10% risk of corrosion)

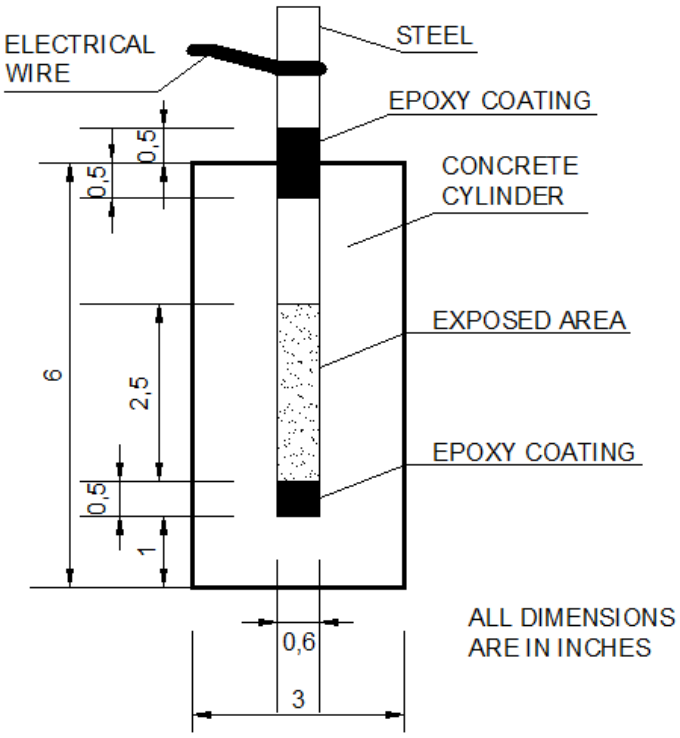

Figure 3-6: Reinforcement Corrosion Specimen.

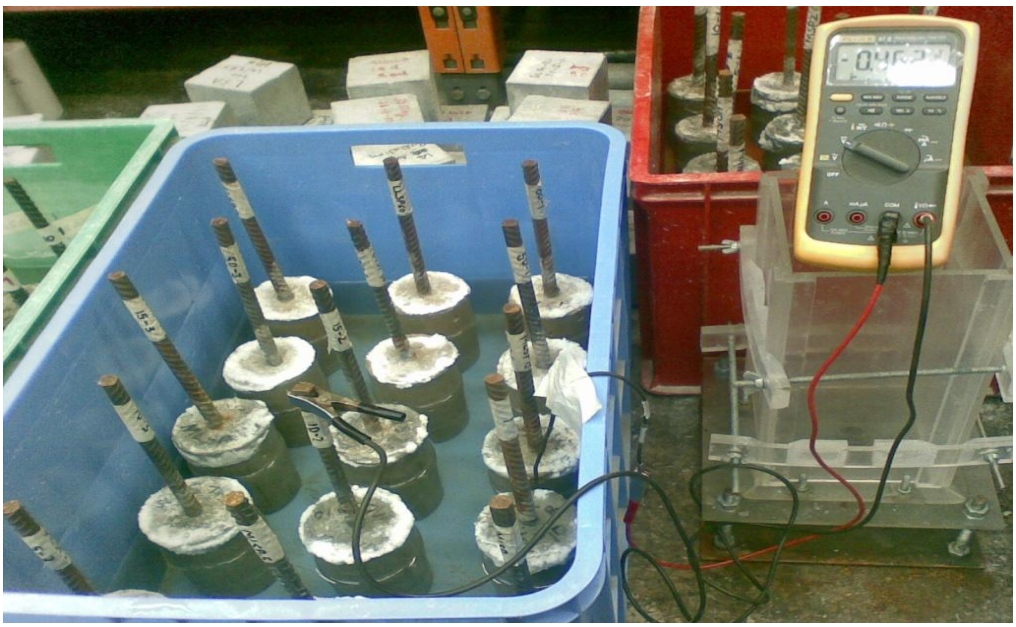

Figure 3-7: Corrosion Potential Measurement Setup.

3.5.6.2 Corrosion Current Density

The corrosion current density measurements provide an indication of the rate at which the reinforcement corrosion is progressing. This information is of great importance in knowing the extent of corrosion damage and in predicting the remaining service life, which is useful in taking decisions regarding the repair and rehabilitation works. Corrosion rate (i.e., corrosion current density) was measured according to the linear polarization resistance method (LPRM) [79].

In LPRM experiments, a stainless steel plate was used as a counter electrode. The steel bar and stainless steel plate was connected to a Potentiostat/Galvanostat. The polarization resistance (R_p) was determined by conducting a linear polarization scan in the range of ±10 mV of the corrosion potential. A scan rate of 0.1 mV/s was used. The corrosion current density (I_{corr}) was determined using the Stern and Geary formula shown below [79]. Schematic representation of the experimental set-up utilized to measure the corrosion current density on steel in the concrete specimens is shown in Figure 3.7 and the experimental set-up is in Figure 3.8. Three specimens were tested and the average I_{corr} values were reported.

$$I_{corr} = B/R_p$$

Where I_{corr} = corrosion current density, $\mu A/cm^2$,

R_p = polarization resistance Ω cm^2,

$B = (\beta a * \beta c)/2.3\ (\beta a + \beta c)$,

βa and βc are the anodic and cathodic Tafel constants, respectively.

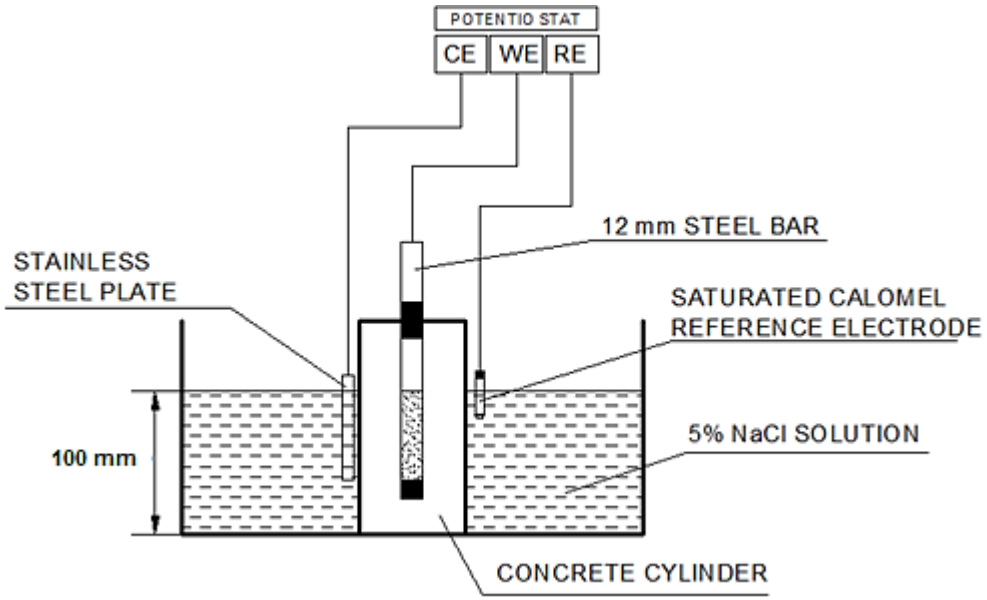

Figure 3-8: Schematic Representation of the Corrosion Current Density Measurements.

Figure 3-9: Corrosion Current Density Setup.

3.5.7 Leachability of Heavy Metals

Leaching is the extraction and measurement of concentration of certain elements that are responsible for any potential contamination. In leaching analysis, the waste component was removed chemically into solution from concrete specimens and then assessed for leachants. Following procedure was utilized to determine the leachability of heavy metals [81].

The leaching of heavy metals was performed according to US EPA Toxicity Characteristic Leaching Procedure (TCLP) [81]. The concrete specimens after 28 days of curing were crushed and sieved through a 1mm sieve. Ten grams of sieved sample were taken in TCLP extraction flask and to that 200 ml of TCLP buffer (1:20) ratio was added. Since the sample pH was greater than 5, the TCLP buffer fluid # 2 was used. Fluid # 2 was prepared by adding 5.7 ml of glacial acid to 500 ml double distilled water (DDW) in 1 L volumetric flask and a pH of 2.88 was recorded. The TCLP extraction flask was then installed in a TCLP rotary apparatus which was made to rotate at 30 rpm for 18 hours. The leachant was then taken out from the extraction flask and filtered using Whatman 42 filter paper. Thereafter, all the samples were acidified to pH 2 using 2% of acetic acid. Then, the samples were analysed by inductively coupled plasma (ICP) apparatus to determine TCLP metals.

3.5.8 Drying Shrinkage

Shrinkage is the reduction in the volume of concrete caused mainly by the loss of water mainly due to evaporation from a freshly hardened concrete exposed to air. Shrinkage may result in cracking of restrained concrete members. Concrete specimens, 40 x 40 x 160 mm were prepared for determining the drying shrinkage according to ASTM C 157 [82]. Three

specimens were tested and the average values are reported. A set of shrinkage specimens with demec gauge and the measuring device are shown in Figure 3.10.

Figure 3-10: Drying Shrinkage Specimens and dial gauge.

CHAPTER 4

RESULTS

The experimental program was discussed in chapter 3. In this chapter, the results of the experimental work for concrete produced using waste materials (CKD, LSP, EAFD, OA and NP) are presented.

4.1 Properties of CKD Cement Concrete Specimens

The various properties of CKD cement concrete are discussed in this section with 5, 10 and 20% CKD, as replacement of cement.

4.1.1 Compressive Strength Development

The concrete produced with waste materials was classified according to following typical strength classification.

Table 4-1: Compressive Strength Classification of Concrete.

Strength classification	**Strength, MPa**
Very low strength	10 - 20
Low strength	20 - 30
Medium strength	30 - 50
High strength	50 - 100
Very high strength	100 - 150
Ultra high strength	> 150

Concrete Specimens with a w/cm Ratio of 0.45

The compressive strength development in the concrete specimens with CKD content varying from 5 to 20%, as replacement of cement, is depicted in Figure 4.1. The compressive strength increased with age in all the concrete specimens. The compressive strength development of 5, 10 and 20% CKD cement concrete was almost similar and slight improvement in strength was noted after 90 days of curing when compared to 0% CKD cement concrete. After 90 days of curing, the compressive strength of 0, 5, 10, and 20% CKD cement concrete specimens was 59.8, 66.1, 64.0, and 63.6 MPa, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The compressive strength development in the concrete specimens with CKD content of 50 and 60%, as replacement of cement, is depicted in Figure 4.2. The compressive strength increased with age in all the concrete specimens. Further, the compressive strength development of 50% NP was more than that of plain cement concrete. The compressive strength development of 50% CKD was higher than that of 60% CKD cement concrete. After 90 days of curing, the compressive strength of 0, 50 and 60% CKD cement concrete specimens was 29.7, 32.7 and 27.4 MPa, respectively.

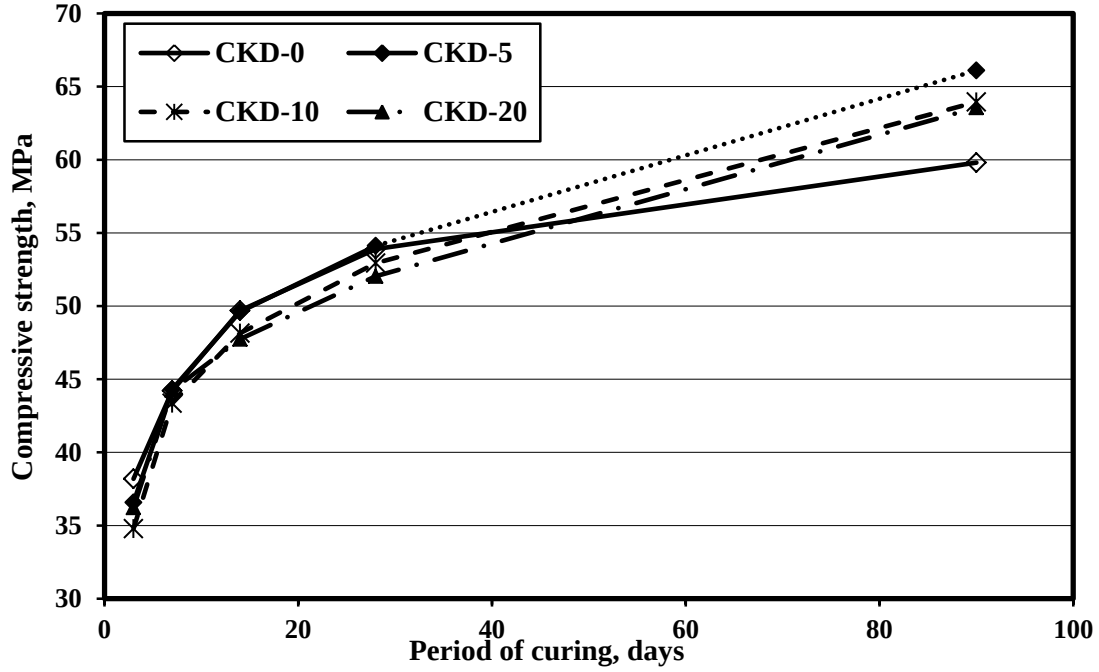

Figure 4-1: Compressive Strength Development of Concrete Specimens Prepared with Different Percentages of CKD and w/cm Ratio of 0.45.

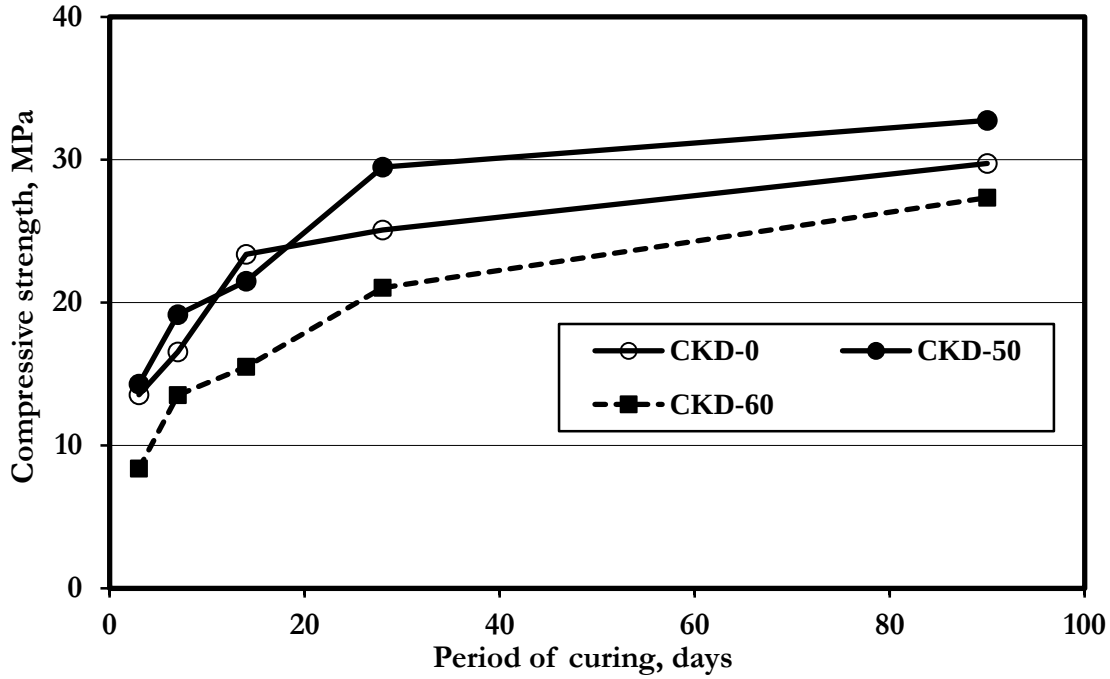

Figure 4-2: Compressive Strength Development of Concrete Specimens Prepared with 50 and 60% CKD and w/cm Ratio of 0.55.

According to the data in Figures 4.1 and 4.2 and in Table 4.1, the results indicate that the CKD content from 5 to 20%, as a replacement of cement, could be utilized as high strength concrete, since the strength was higher than 50 MPa while concretes with 50 and 60% CKD can be utilized as medium strength concrete.

4.1.2 Water Absorption

Concrete Specimens with a w/cm Ratio of 0.45

The water absorption in the concrete specimens after 28 days of water curing with CKD varying from 0 to 20%, as a replacement of cement, is depicted in Figure 4.3. The water absorption increased with an increase in the quantity of CKD. The water absorption of 0, 5, 10, and 20% CKD cement concrete was 4.1, 4.2, 4.6, and 4.8%, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The water absorption in the concrete specimens with CKD content of 50 and 60%, as a replacement of cement, is depicted in Figure 4.4. The water absorption increased with an increase in the CKD content. The water absorption of 50% and 60% CKD cement concrete specimens was more than that of plain cement concrete. After 28 days of curing, the water absorption of 0, 50 and 60% CKD cement concrete specimens was 5.3, 5.6, and 6.2%, respectively.

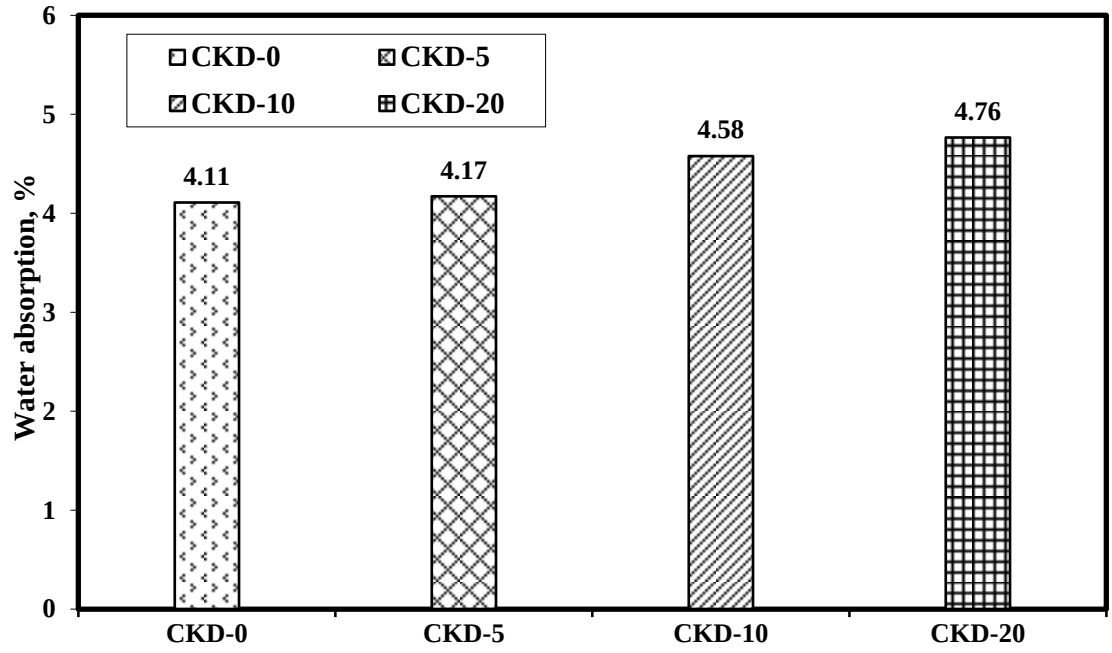

Figure 4-3: Water Absorption of Concrete Specimens Prepared with Different Percentages of CKD and a w/cm Ratio of 0.45.

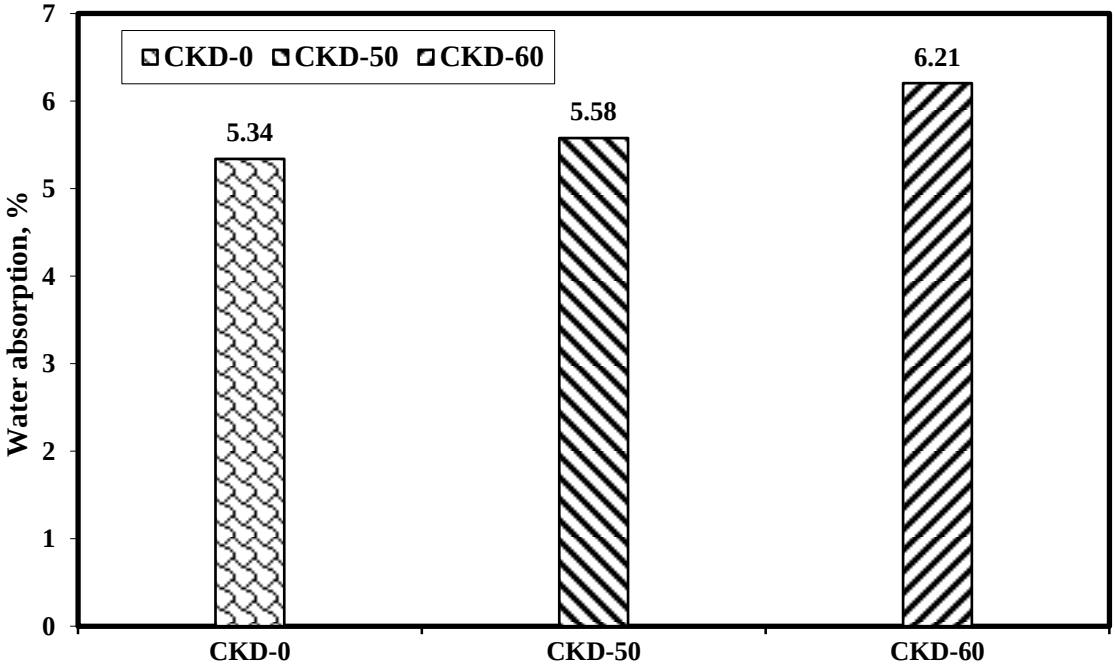

Figure 4-4: Water Absorption of Concrete Specimens Prepared with 50 and 60% CKD and a w/cm Ratio of 0.55.

4.1.3 Corrosion Potentials

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion potentials on steel in the concrete specimens prepared using CKD content varying from 0 to 20%, as replacement of cement, are depicted in Figure 4.5. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the reinforced concrete specimens. The corrosion potentials for the 20% CKD cement concrete specimens were more than those in the 0% CKD cement concrete after 200 days of exposure. The corrosion potentials on steel in the concrete specimens with 5 and 10% CKD cement concrete were less than that of 0% CKD cement concrete. The time to initiation of corrosion (i.e., to cross the -270 mV potential) was about 58, 76, 18, and 5 days for 0, 5, 10, and 20% CKD cement concrete, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The corrosion potentials on steel in the concrete specimens prepared with CKD content 50 and 60%, as replacement of cement, are depicted in Figure 4.6. The potentials were more negative than the ASTM C 876 threshold value of -270 mV SCE since the time of immersion in the chloride solution. Further, the potentials in both the specimens were almost the same.

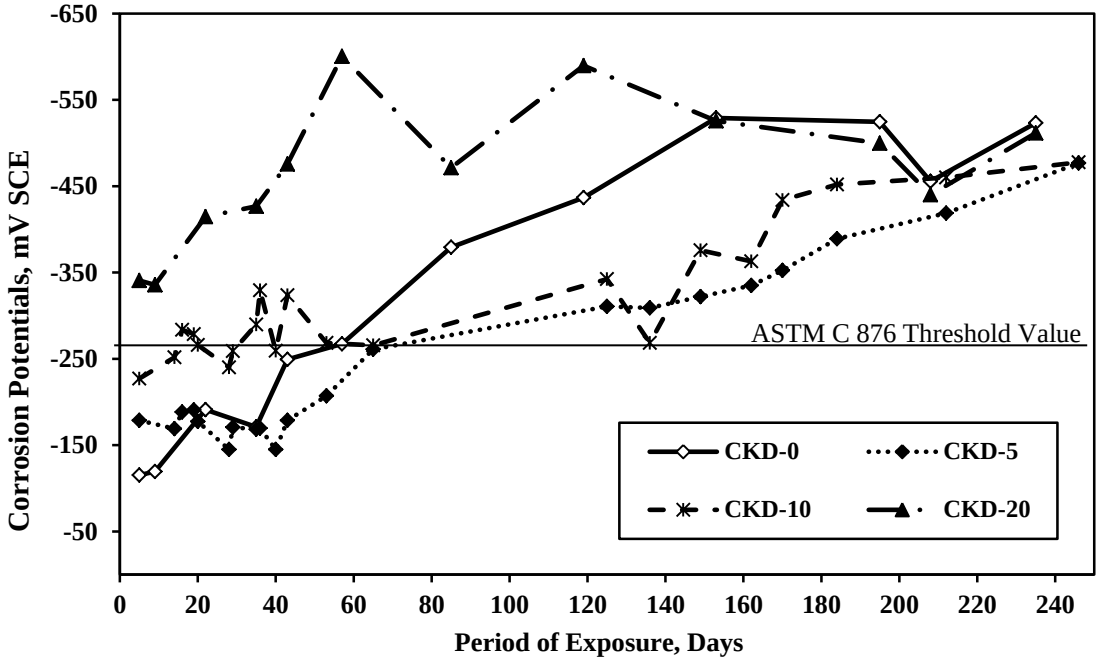

Figure 4-5: Corrosion Potentials on Steel in the Concrete Specimens Prepared with Different Percentages of CKD and a w/cm Ratio of 0.45.

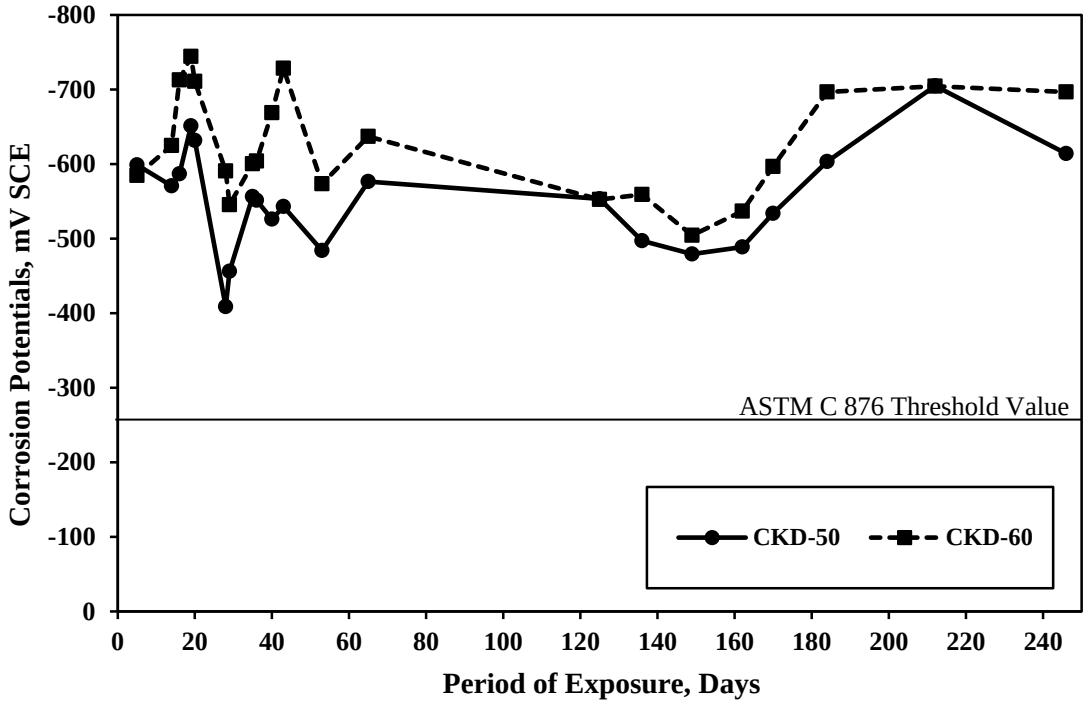

Figure 4-6: Corrosion Potentials on Steel in the Concrete Specimens Prepared with 50 and 60% CKD and a w/cm Ratio of 0.55.

4.1.4 Corrosion Current Density

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion current density (I_{corr}) on steel in the concrete specimens prepared using CKD content varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.7. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. The I_{corr} for 5, 10 and 20% CKD cement concrete specimens was less than that on steel in the 0% CKD cement concrete specimens. After 5 months exposure, the I_{corr} values on the 0, 5, 10 and 20% CKD cement concrete are 0.63, 0.51, 0.56 and 0.55 respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The I_{corr} on steel in the concrete specimens prepared with 50 and 60% CKD, as a replacement of cement, is depicted in Figure 4.8. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. The I_{corr} in 50% CKD concrete specimens was more than that on 60% CKD cement concrete specimens for all the exposure periods. After 5 months exposure, the I_{corr} values are 0.71 and 0.65 for the 50 and 60% CKD, respectively.

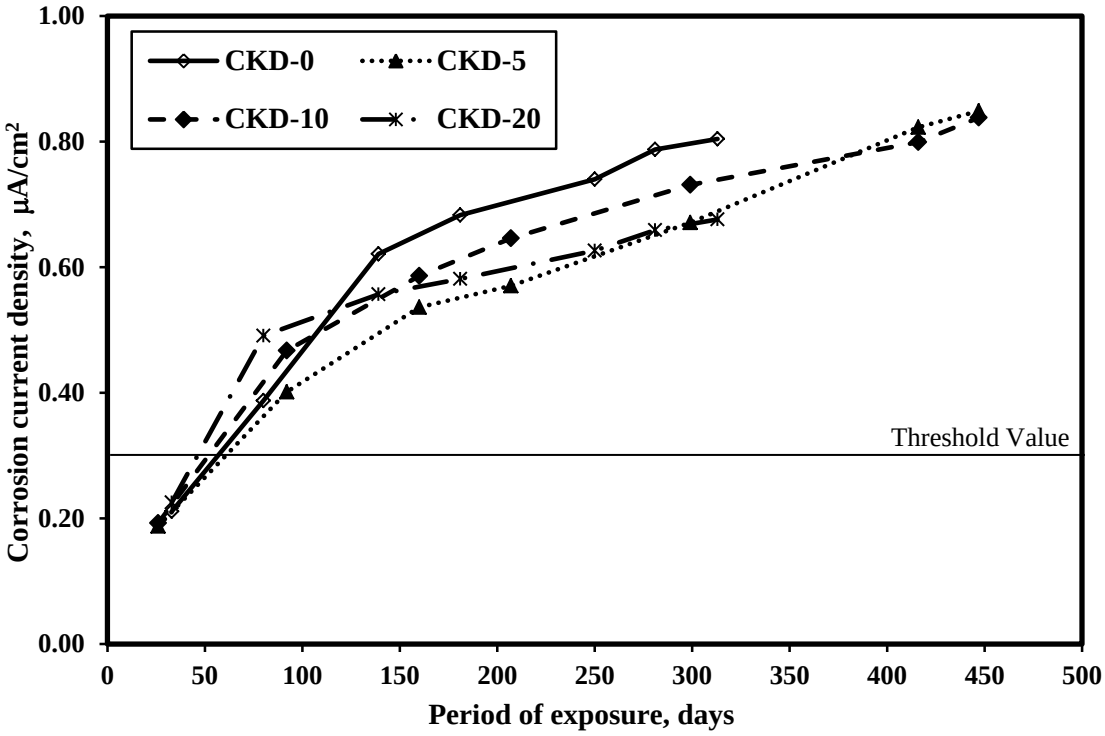

Figure 4-7: Corrosion Current Density on Steel in the Concrete Specimens prepared with Different Percentages of CKD and a w/cm Ratio of 0.45.

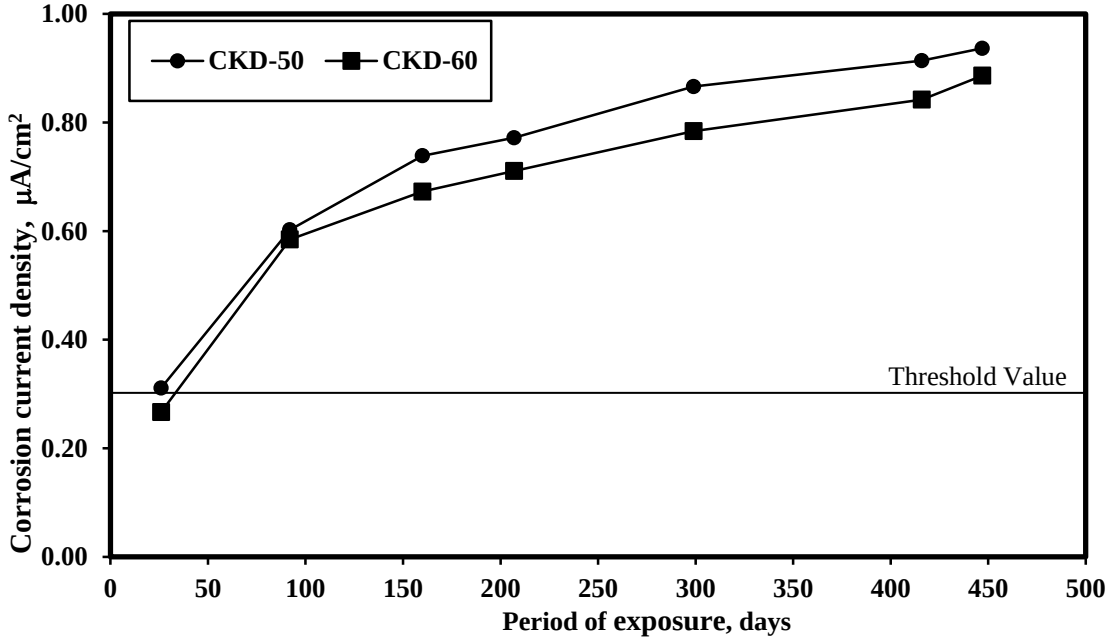

Figure 4-8: Corrosion Current Density on Steel in the Concrete Specimens prepared with 50 and 60% CKD and a w/cm Ratio of 0.55.

4.1.5 Electrical Resistivity

Concrete Specimens with a w/cm Ratio of 0.45

The electrical resistivity of concrete specimens prepared with CKD content varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.9 after 28 days of curing. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. However, the electrical resistivity did not decrease much after a moisture content of 2.5%. The electrical resistivity at 3% moisture content for 0, 5, 10 and 20% CKD cement concrete was 34.2, 33.3, 29, and 21.2 kΩ-cm, respectively. With these results, it is observed that electrical resistivity decreased with an increase in CKD content, and with regard to risk of reinforcement corrosion summarised in Table 3.8, the specimens are in the range of moderate intensity. The electrical resistivity of concrete decreases both due to the presence of moisture and chloride ions [33].

Concrete Specimens with a w/cm Ratio of 0.55

The electrical resistivity of concrete specimens prepared using 50 and 60% CKD, as replacement of cement, is depicted in Figure 4.10 after 28 days of curing. The electrical resistivity decreased with an increase in the moisture content. The electrical resistivity at 3% moisture content for 50 and 60% CKD cement concrete was 17.74 and 15.15 kΩ-cm, respectively. With these values, the risk of reinforcement corrosion in these specimens is moderate (Table 3.8).

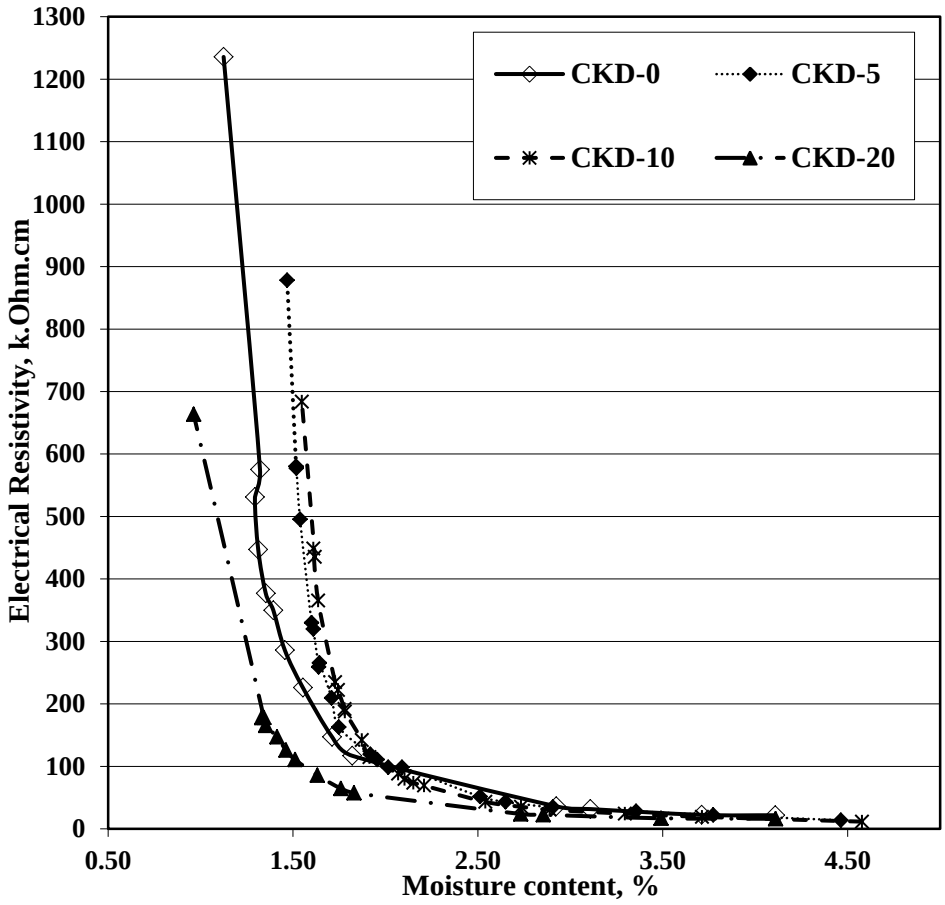

Figure 4-9: Electrical Resistivity of Concrete Specimens prepared with Different Percentages of CKD and a w/cm Ratio of 0.45.

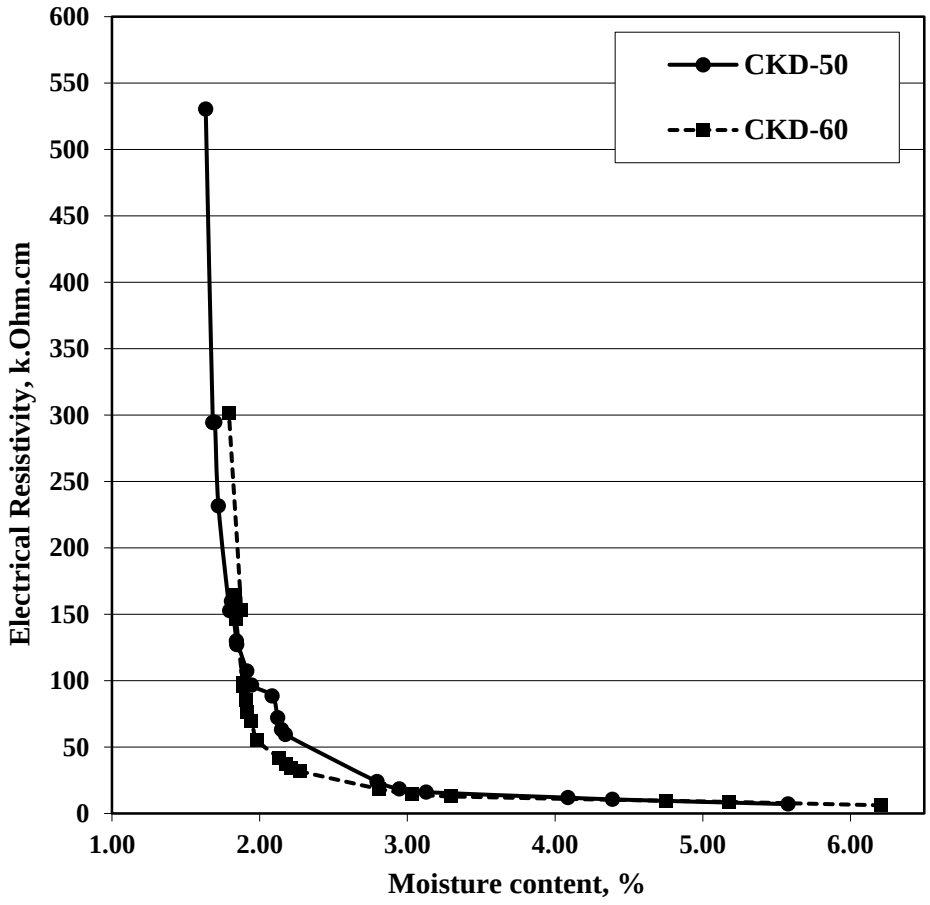

Figure 4-10: Electrical Resistivity of Concrete Specimens prepared with 50 and 60% CKD and a w/cm Ratio of 0.55.

4.1.6 Thermal Conductivity

Earlier studies revealed that the thermal conductivity of mortar and cement paste was affected by types of admixtures, amount of cement, fraction of aggregate volume and moisture condition of concrete. It was also reported that the thermal conductivity of concrete is hardly affected by its age [83].

Concrete Specimens with a w/cm Ratio of 0.45

The thermal conductivity in the concrete specimens with CKD varying from 0 to 20%, as a replacement of cement, is depicted in Figure 4.11. The thermal conductivity increased with an increase in the quantity of CKD. However, the increase in the thermal conductivity with increasing CKD content was not that significant. After 28 days of curing, the thermal conductivity of 0, 5, 10, and 20% CKD cement concrete was 2.4, 2.6, 2.6, and 2.6 W/mK, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The thermal conductivity in the concrete specimens with CKD 50 and 60%, as replacement of cement, is depicted in Figure 4.12. The thermal conductivity for 60% CKD content was found to be less than that of 50% CKD content. After 28 days of curing, the thermal conductivity of 50 and 60% CKD cement concrete was 2.5, and 2.1 W/mK, respectively.

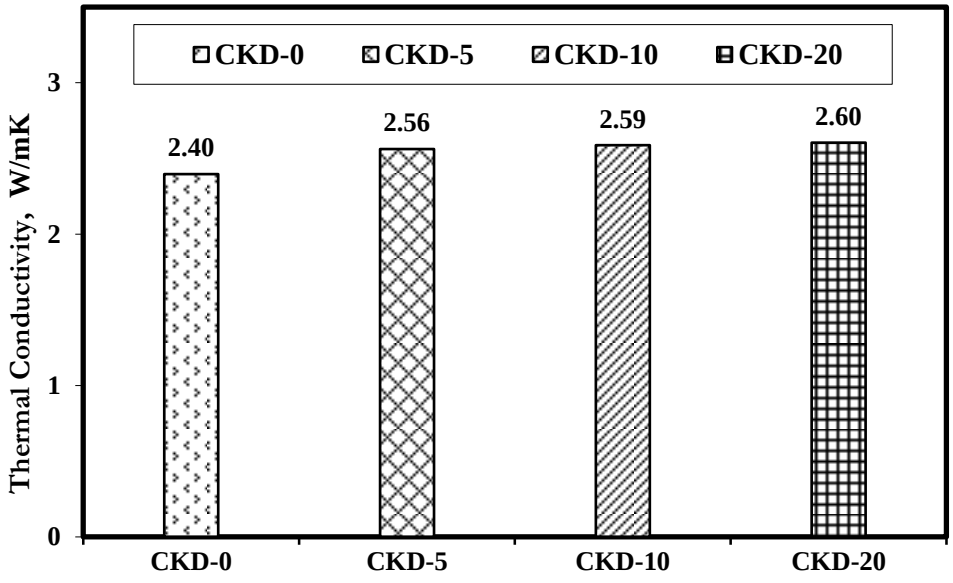

Figure 4-11: Thermal Conductivity of Concrete Specimens prepared with Different Percentages of CKD and a w/cm Ratio of 0.45.

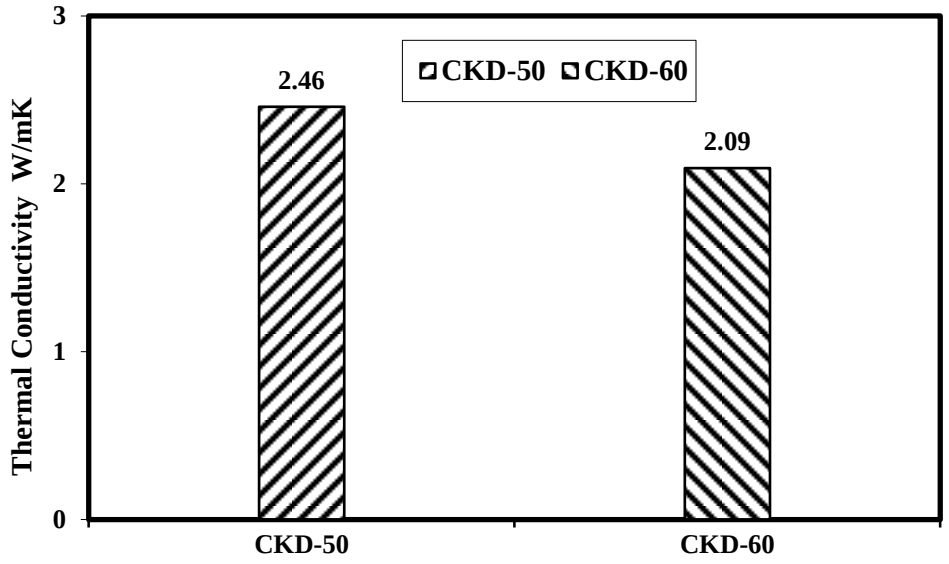

Figure 4-12: Thermal Conductivity of Concrete Specimens Prepared with50 and 60% CKD and a w/cm Ratio of 0.55.

4.1.7 Drying Shrinkage

Concrete Specimens with a w/cm Ratio of 0.45

The drying shrinkage in concrete specimens prepared with CKD content varying from 0 to 20%, is depicted in Figure 4.13. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, stabilizing with time and remaining almost unchanged thereafter. Maximum drying shrinkage was noted in the mix with 20% CKD. The drying shrinkage in the 5 and 10% CKD cement concrete specimens was less than those specimens without CKD. After 180 days, the drying shrinkage of specimens with 0, 10 and 20% CKD cement concrete was 532, 404, 507, and 557 microns, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The drying shrinkage in concrete specimens prepared with 50 and 60% CKD, is depicted in Figure 4.14. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, stabilizing with time and remaining almost unchanged thereafter. The drying shrinkage in the 50% CKD specimens was less than that in the 60% CKD specimens. After 180 days, the drying shrinkage of 50 and 60% CKD specimens was 545 and 627 microns, respectively.

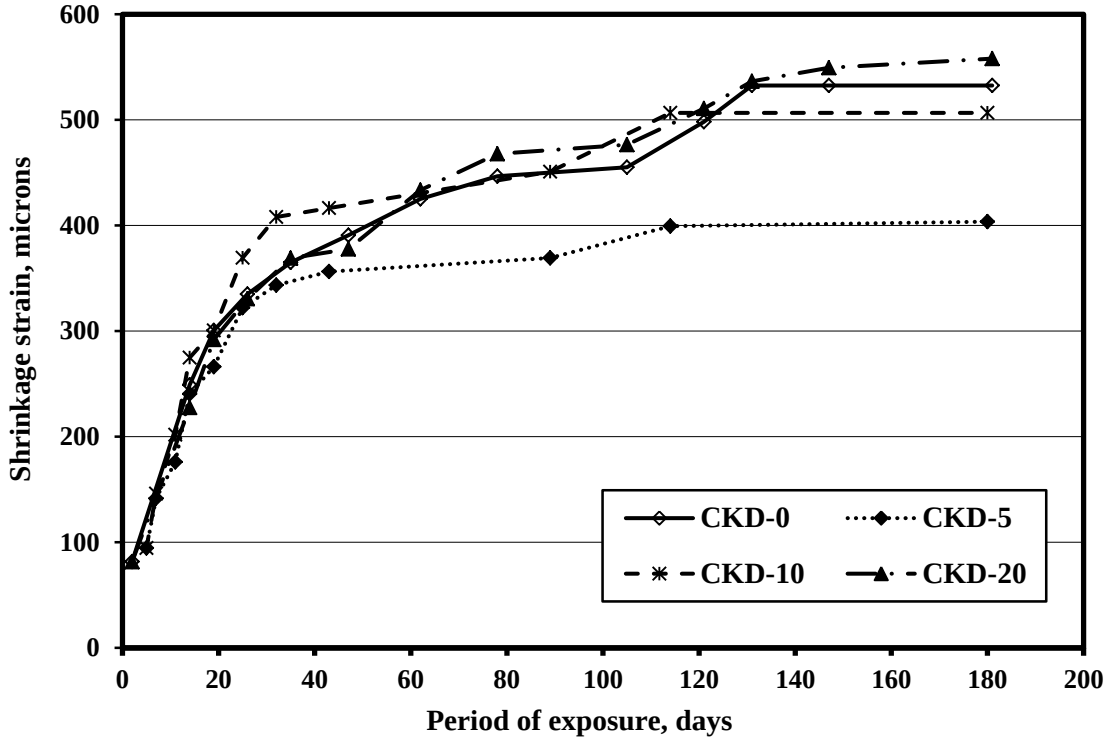

Figure 4-13: Drying Shrinkage Strain in Concrete Specimens Prepared with Different Percentages of CKD and a w/cm of 0.45.

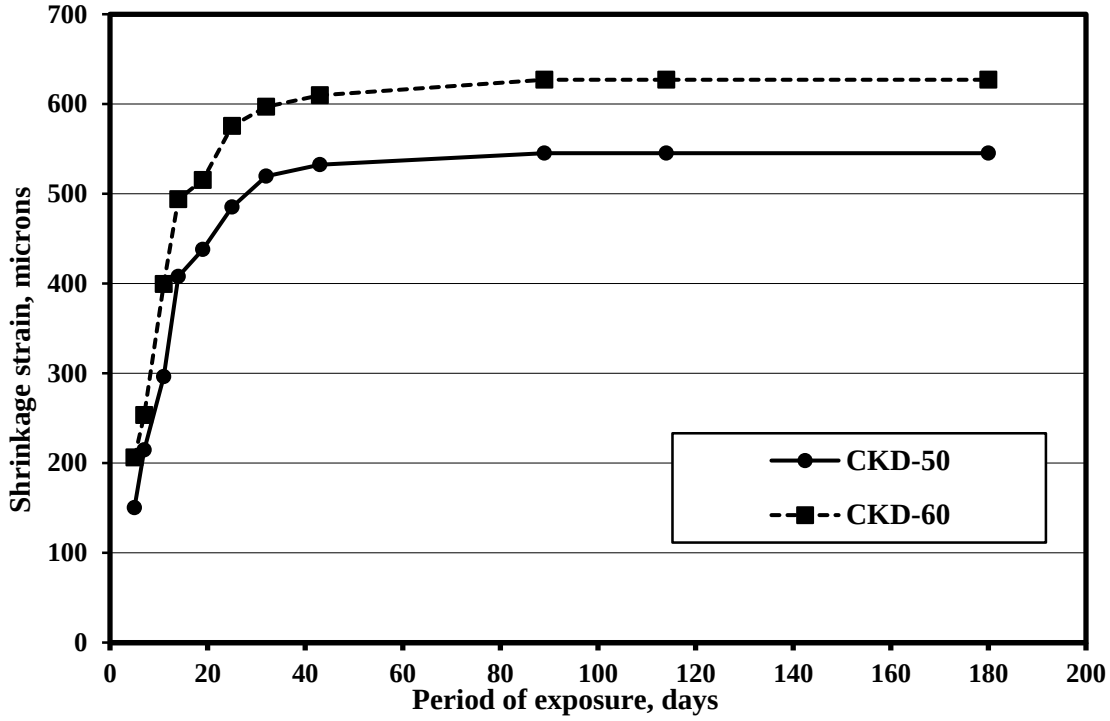

Figure 4-14: Drying Shrinkage Strain in Concrete Specimens Prepared with 50 and 60% CKD and a w/cm Ratio of 0.55.

4.2 Properties of LSP Cement Concrete Specimens

The various properties of LSP cement concrete are discussed in this section with 5, 10 and 15% LSP, as replacement of cement.

4.2.1 Compressive Strength Development

Concrete Specimens with a w/cm Ratio of 0.45

The compressive strength development of the concrete specimens prepared with LSP content varying from 5 to 15%, as replacement of cement, is depicted in Figure 4.15. The compressive strength increased with age for all the concrete mixtures. Further, the compressive strength tended to decrease with an increase in the LSP content. The

compressive strength development of 0 and 10% LSP cement concrete was almost similar while strength of 5% LSP cement concrete was more than 0% CKD cement concrete, after 90 days curing. However, the compressive strength of specimen with 15% LSP was less than that of control specimen. After 90 days of curing, the compressive strength of 0, 5, 10, and 15% LSP cement concrete specimens was 59.8, 63.8, 60.1, and 58.1 MPa, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The compressive strength development in the concrete specimens with LSP content of 50 and 60%, as replacement of cement, is depicted in Figure 4.16. The compressive strength increased with age in all the concrete specimens. Further, the compressive strength tended to decrease with an increase in the LSP content. However, the compressive strength development of 50% LSP was more than that of 60% LSP cement concrete. After 90 days of curing, the compressive strength of 0, 50 and 60% LSP cement concrete specimens was 29.7, 25.8 and 22.0 MPa, respectively.

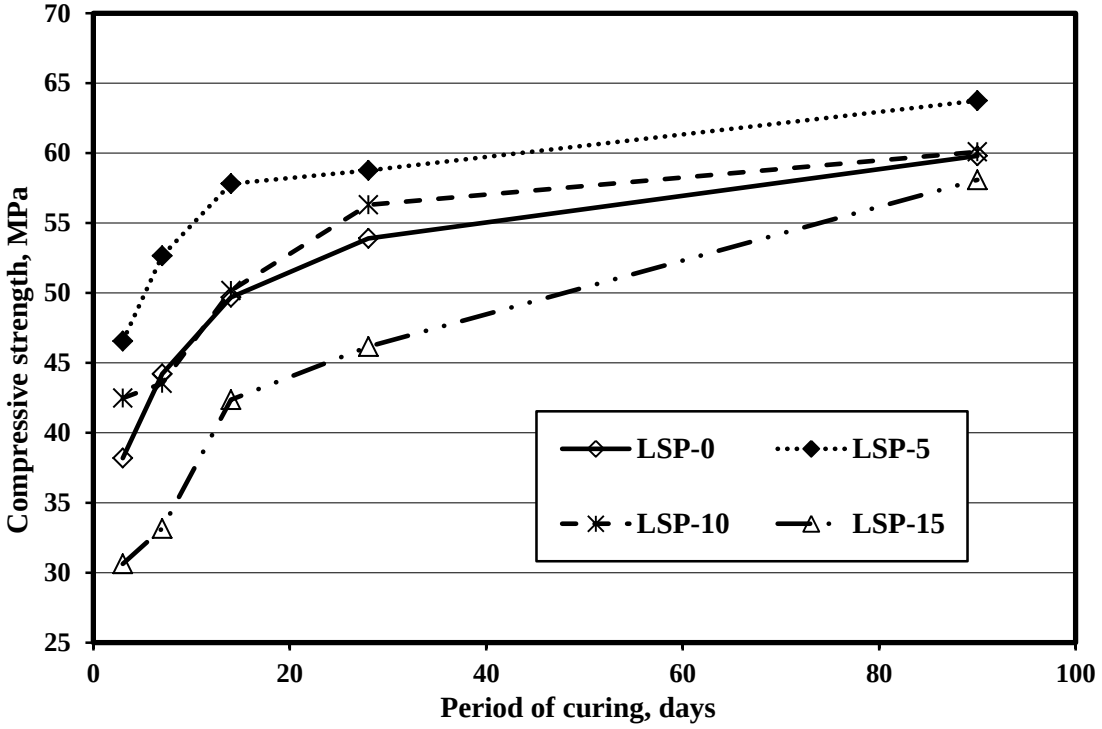

Figure 4-15: Compressive Strength of Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

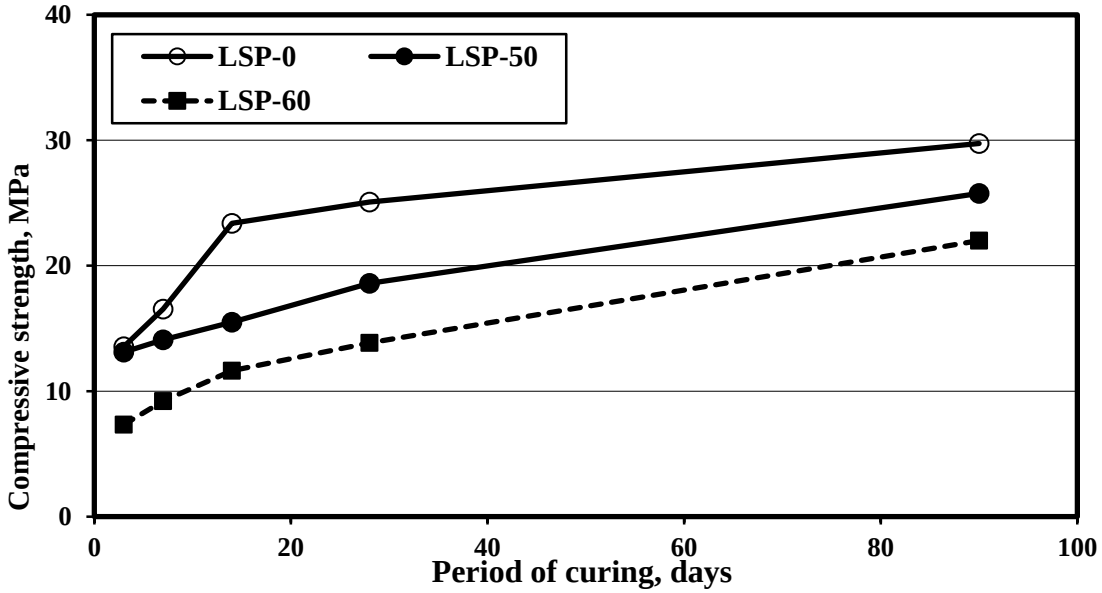

Figure 4-16: Compressive Strength of Concrete Specimens Prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

Referring the data in Figures 4.15 and 4.16 and in Table 4.1, the results indicate that LSP content from 5 to 10%, as a replacement of cement, can be utilized as high strength concrete since the strength was more than 50 MPa, while concrete with 15% LSP content can be used as medium strength concrete since strength was more than 30 MPa. The concretes with 50 and 60% LSP can be utilized as very low strength concrete.

4.2.2 Water Absorption

Concrete Specimens with a w/cm Ratio of 0.45

The water absorption in the concrete specimens with LSP varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.17. The water absorption increased with an increase in the quantity of LSP. However, the water absorption in the concrete specimen with 5% LSP was almost similar to that of the control specimen. The water absorption for 10 and 15% was higher than that of 0% LSP, while the water absorption for 10 and 15% LSP was almost the same. After 28 days of curing, the water absorption of 0, 5, 10, and 15% LSP cement concrete was 4.1, 4.2, 4.7, and 4.7%, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The water absorption in the concrete specimens with 50 and 60% LSP, as replacement of cement, is depicted in Figure 4.18. The water absorption for 50 and 60% LSP was more than that of 0% LSP cement concrete. However, the water absorption in 60% LSP specimen was marginally more than 50% LSP. After 28 days of curing, the water absorption of 0, 50 and 60% LSP cement concrete specimens was 5.3, 6.0, and 6.3%, respectively.

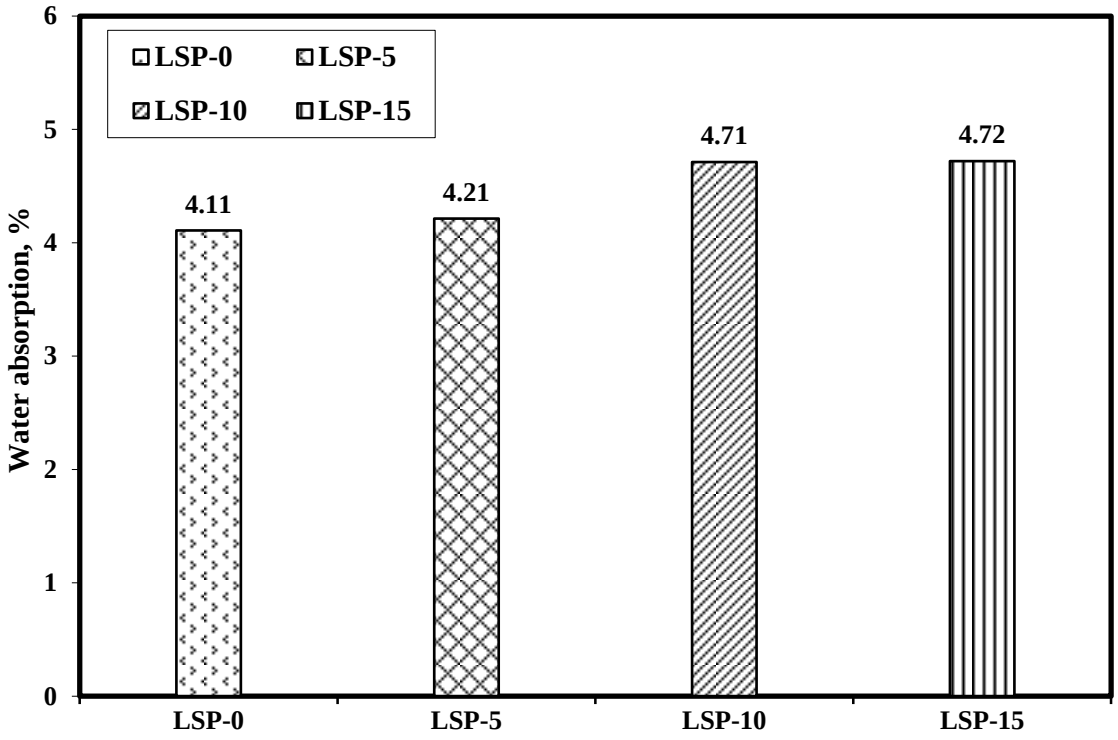

Figure 4-17: Water Absorption of Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

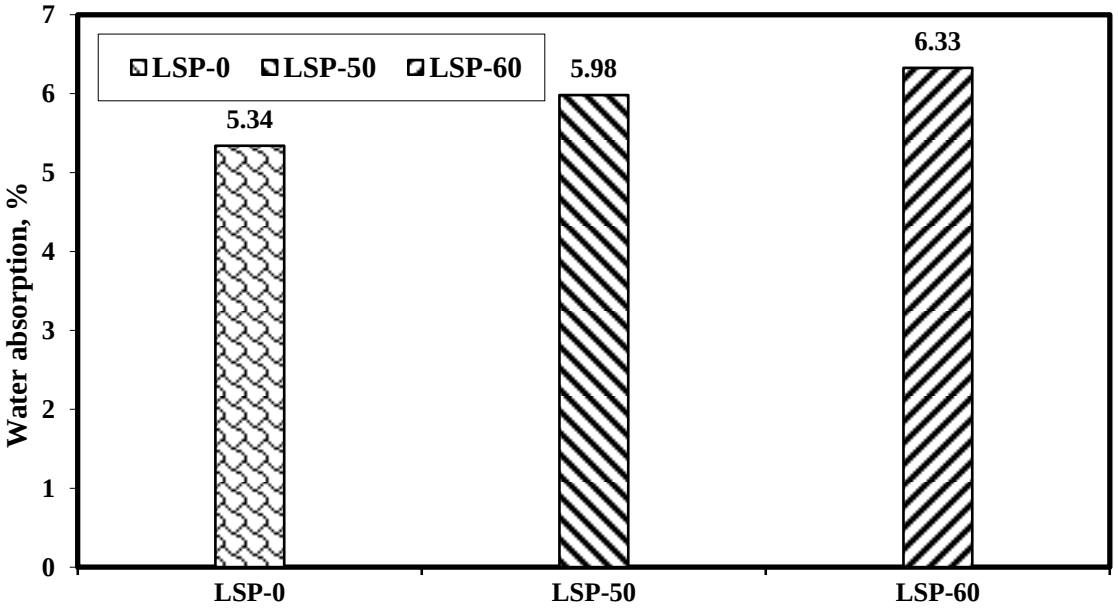

Figure 4-18: Water Absorption of Concrete Specimens Prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

4.2.3 Corrosion Potentials

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion potentials on steel in the concrete specimens prepared with LSP content varying from 0 to 15%, as replacement of cement, are depicted in Figure 4.19. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the reinforced concrete specimens. Further, the variation in potentials was not that significant after 5 months of exposure. The time to initiation of corrosion was about 58, 75, 15, and 23 days for 0, 5, 10, and 15% LSP cement concrete, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The corrosion potentials on steel in the concrete specimens prepared using 50 and 60% LSP, as replacement of cement, are depicted in Figure 4.20. The corrosion potentials were more negative than the threshold value of -270 mV SCE from the beginning of the exposure.

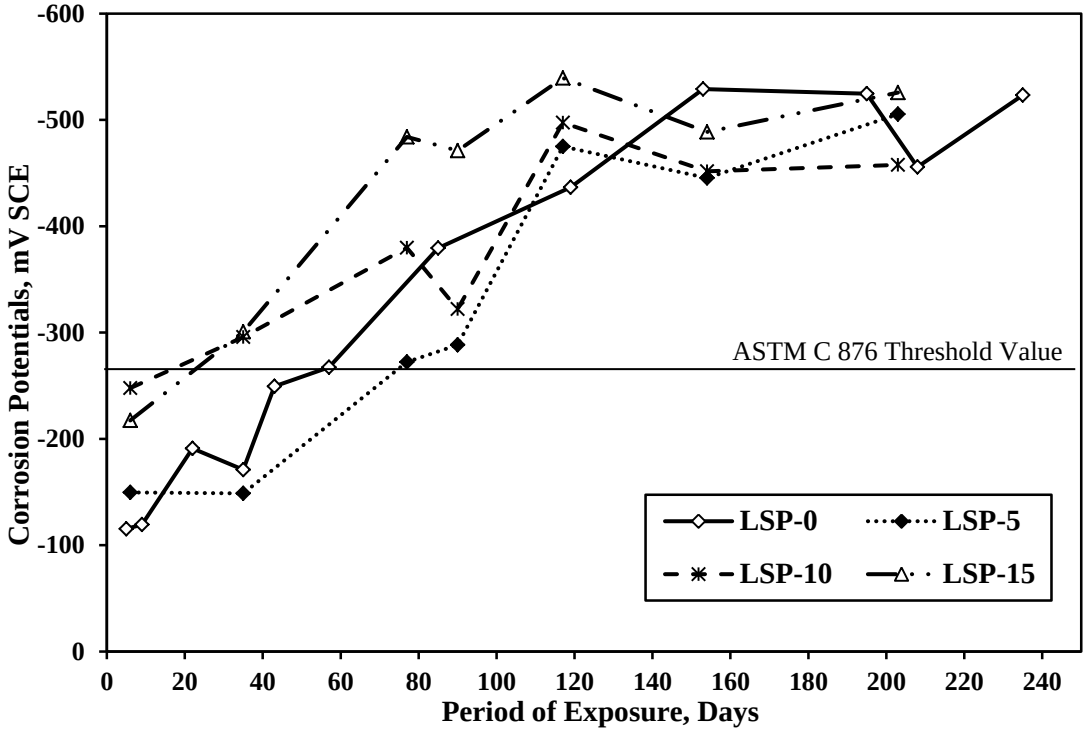

Figure 4- 19: Corrosion Potentials on Steel in the Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

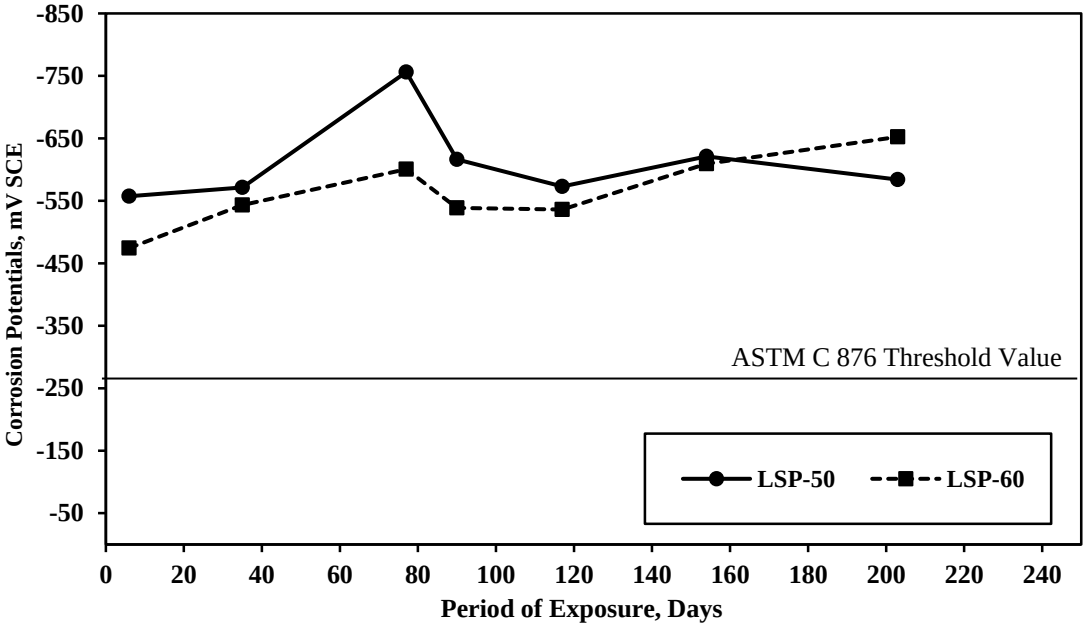

Figure 4-20: Corrosion Potentials on Steel in the Concrete Specimens Prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

4.2.4 Corrosion Current Density

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion current density (I_{corr}) on steel in the concrete specimens prepared using LSP content varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.21. The I_{corr} increased with the period of exposure to the chloride solution in all the concrete specimens. The I_{corr} in all the LSP concrete specimens was less than that on steel in the concrete specimen with 0% LSP. It can be observed from the data in Figure 4.21 that I_{corr} tended to increase with the increase n LSP. After about 200 days the I_{corr} was 0.70, 0.39, 0.67 and 0.62 $\mu A/cm^2$, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The I_{corr} on steel in the concrete specimens prepared using 50 and 60% LSP, as replacement of cement, is depicted in Figure 4.22. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. The I_{corr} on steel in the concrete specimens prepared with 50% LSP was more than that on specimens prepared with 60% LSP. After 200 days, the I_{corr} was 0.73 and 0. 69 $\mu A/cm^2$, respectively.

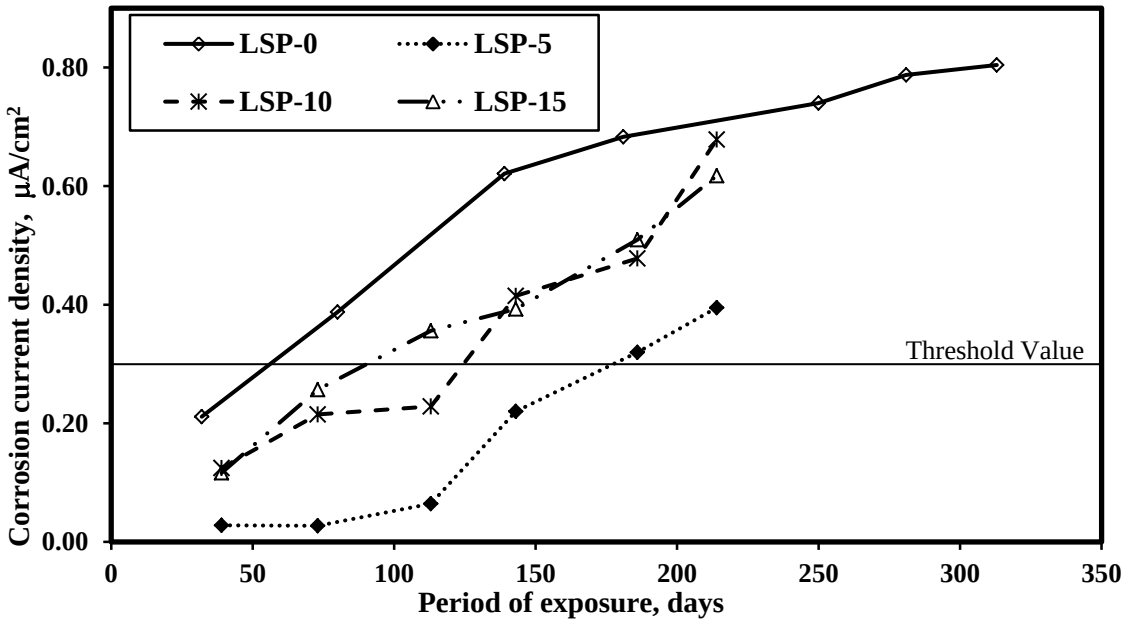

Figure 4-21: Corrosion Current Density on Steel in the Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

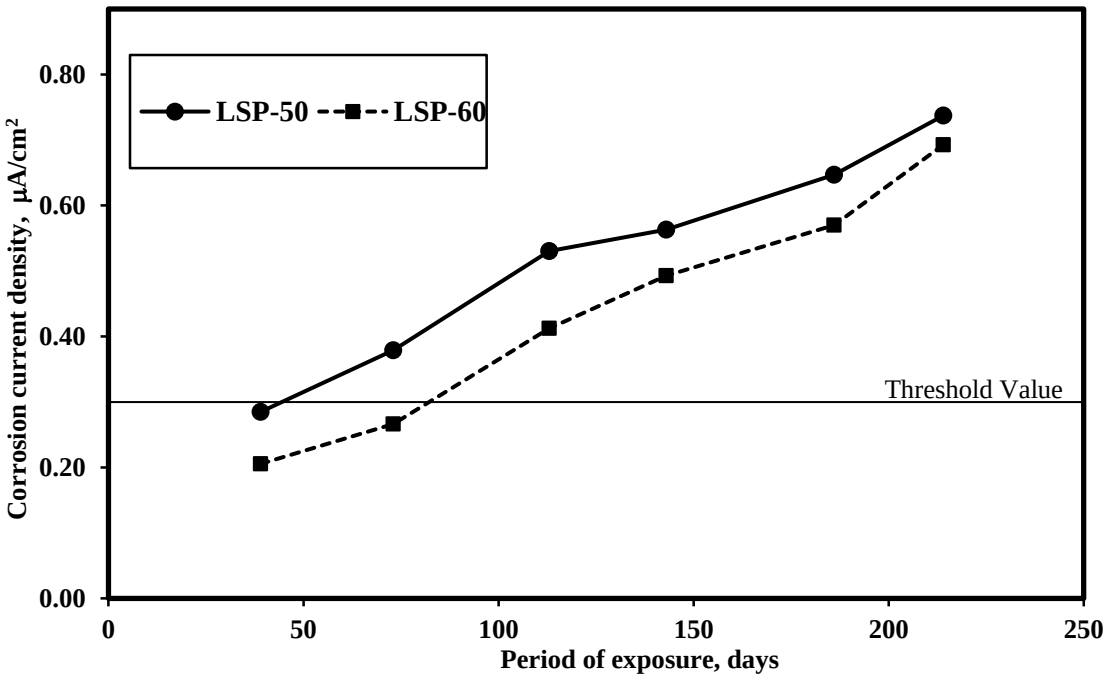

Figure 4-22: Corrosion Current Density on Steel in the Concrete Specimens Prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

4.2.5 Electrical Resistivity

Concrete Specimens with a w/cm Ratio of 0.45

The electrical resistivity of concrete specimens prepared using LSP content varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.23. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. However, it did not decrease any further after about 3% moisture content. The electrical resistivity was almost same in all the specimens. The electrical resistivity at 3% moisture content in 0, 5, 10 and 15% LSP cement concrete specimens was 34.2, 63.2, 65.9, and 63.7 kΩ-cm, respectively. With regard to the risk of reinforcement corrosion summarised in Table 3.8, the specimens are in the low category.

Concrete Specimens with a w/cm Ratio of 0.55

The electrical resistivity of concrete specimens prepared with 50 and 60% LSP, as replacement of cement, is depicted in Figure 4.24. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. The electrical resistivity at 3% moisture content for 50 and 60% LSP cement concrete was 102.4 and 76.15 kΩ-cm, respectively. With these results, it is observed that the electrical resistivity decreased with an increase in the LSP content, and with regard to risk of reinforcement corrosion summarised in Table 3.8, the specimen with 50% LSP is in the range of negligible intensity and 60% LSP is in the range of low intensity.

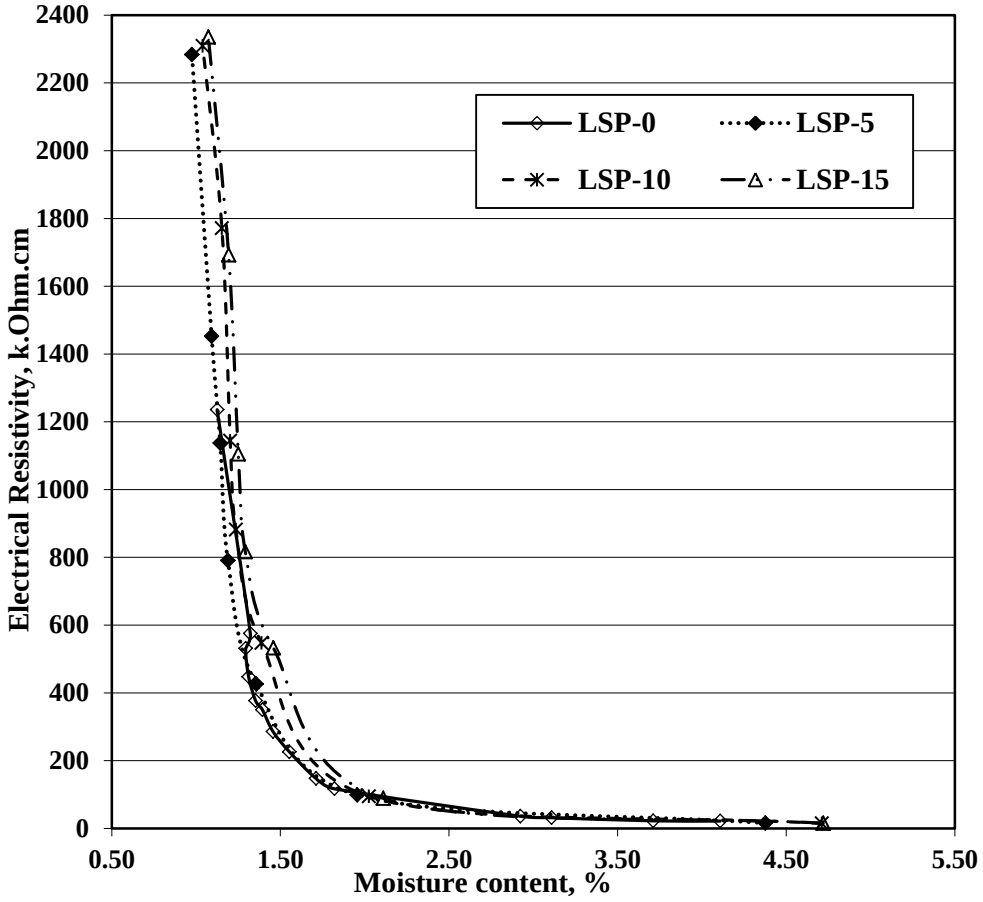

Figure 4-23: Electrical Resistivity of Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

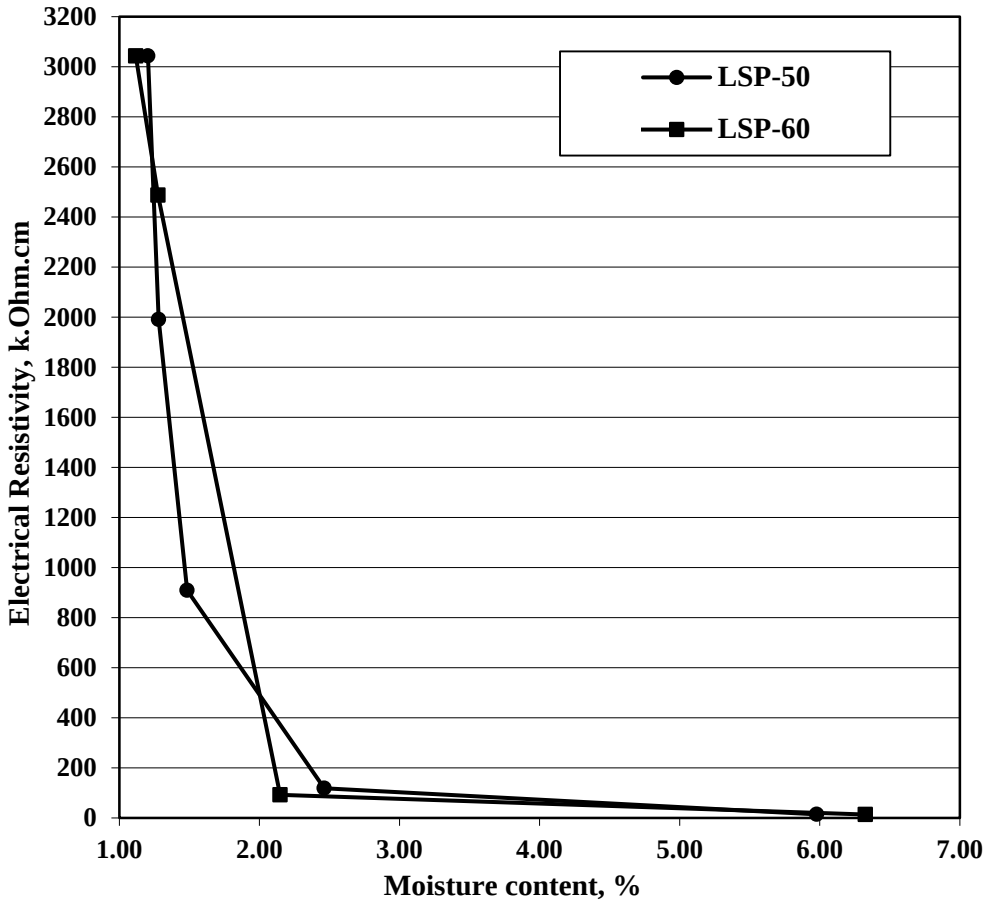

Figure 4-24: Electrical Resistivity of Concrete Specimens Prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

4.2.6 Thermal Conductivity

Concrete Specimens with a w/cm Ratio of 0.45

The thermal conductivity in the concrete specimens with LSP varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.25. Thermal conductivity for 5, 10 and15% LSP cement concrete specimens was almost same and is more than that of 0% LSP cement

concrete. After 28 days of curing, the thermal conductivity of 0, 5, 10, and 15% LSP was 2.4, 2.69, 2.67, and 2.64 W/mk, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The thermal conductivity of the concrete specimens with 50 and 60% LSP, as replacement of cement, is depicted in Figure 4.26. The thermal conductivity of 60% LSP was less than that of 50% LSP cement concrete. After 28 days of curing, the thermal conductivity of 50 and 60% LSP cement concrete specimens was 2.96, and 2.77 W/mk, respectively.

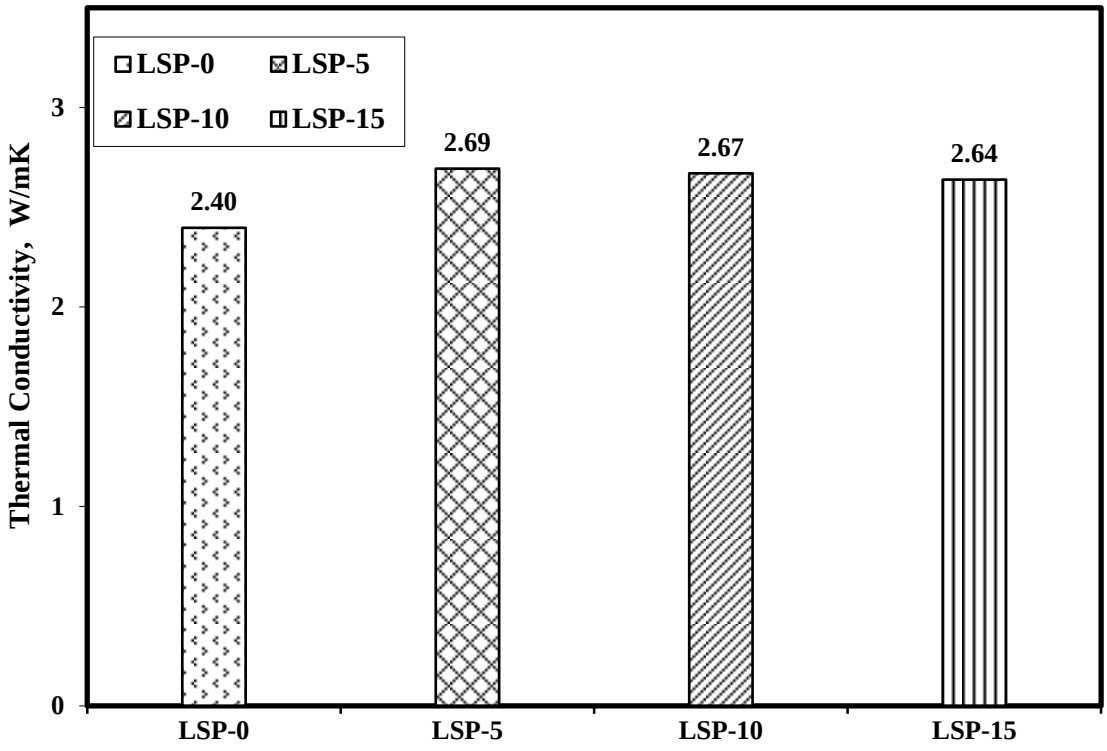

Figure 4-25: Thermal Conductivity of Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

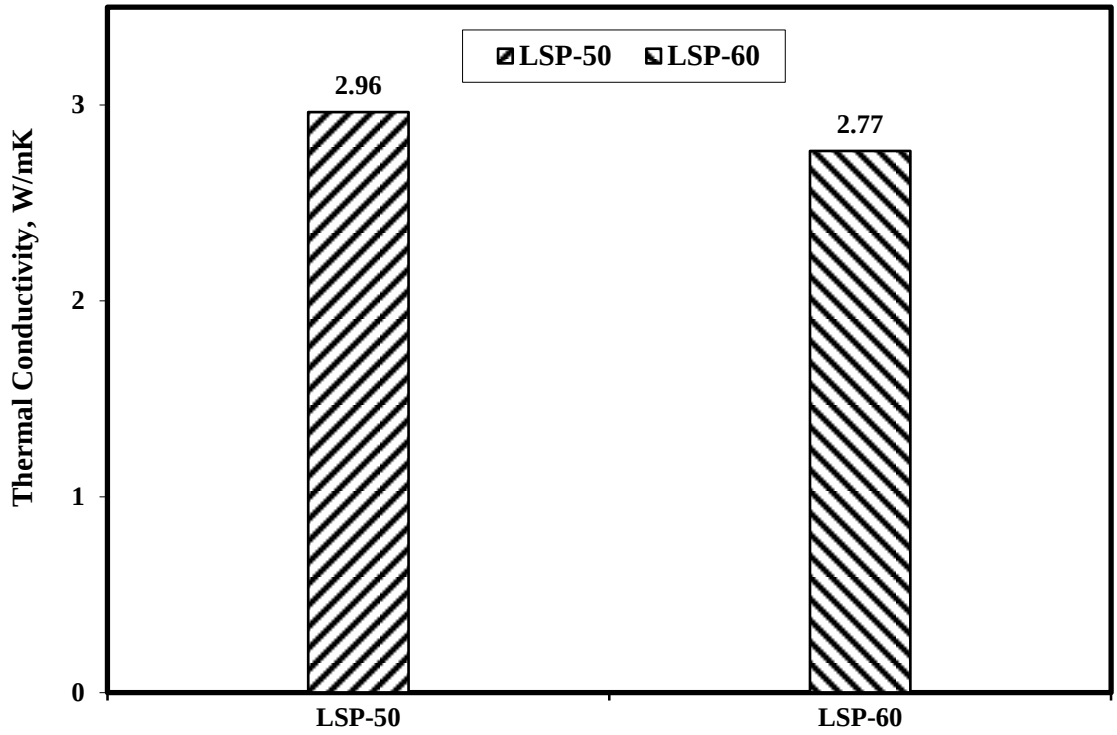

Figure 4-26: Thermal Conductivity of Concrete Specimens Prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

4.2.7 Drying Shrinkage

Concrete Specimens with a w/cm Ratio of 0.45

The drying shrinkage in concrete specimens prepared using LSP content varying from 0 to 15% is depicted in Figure 4.27. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially and stabilized with time. The maximum drying shrinkage occurred in mix with 15% LSP content. Drying shrinkage with 10% LSP content was a little more than that of 0% LSP cement concrete. At 180 days, the drying

shrinkage of specimens with 0, 5, 10 and 15% LSP cement concrete was 532.5, 519.3, 545.2, and 609.6 microns, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The drying shrinkage in the concrete specimens prepared using 50 and 60% LSP, is depicted in Figure 4.28. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, thereafter, it stabilized with time. The drying shrinkage of 50% LSP specimens was less than that in 60% LSP specimens. At 180 days, the drying shrinkage of specimens 50 and 60% LSP cement concrete specimens was 764 and 785 microns, respectively.

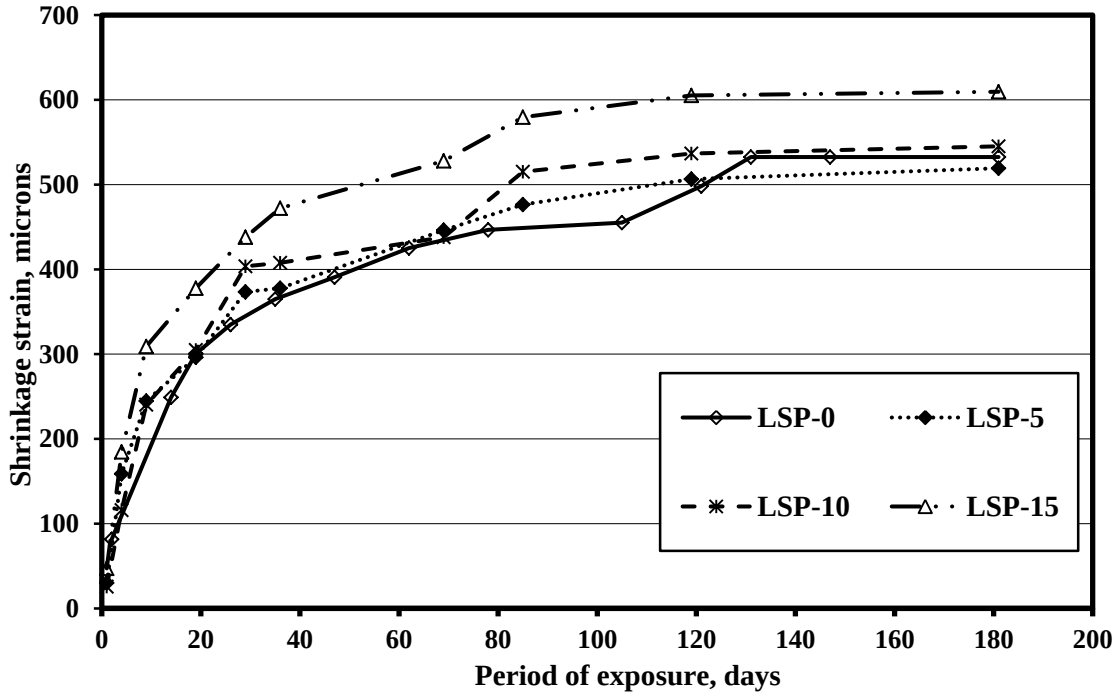

Figure 4-27: Drying Shrinkage Strain in Concrete Specimens Prepared with Different Percentages of LSP and a w/cm Ratio of 0.45.

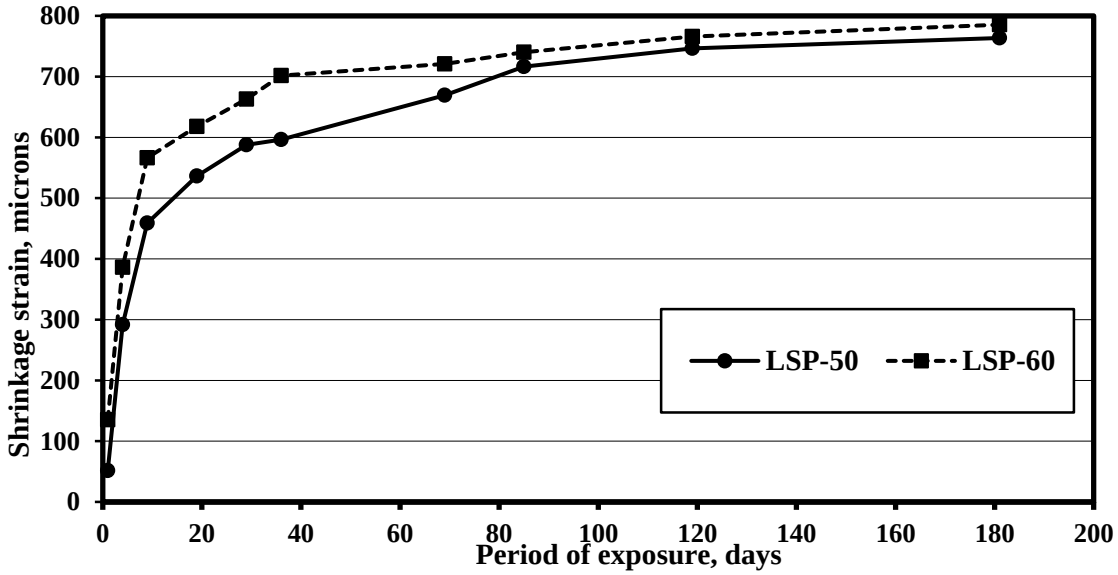

Figure 4-28: Drying Shrinkage Strain in Concrete Specimens prepared with 50 and 60% LSP and a w/cm Ratio of 0.55.

4.3 Properties of EAFD Cement Concrete Specimens

The various properties of EAFD cement concrete are discussed in this section with 5, 10 and 15% EAFD, as replacement of cement.

4.3.1 Compressive Strength Development

Concrete Specimens with a w/cm Ratio of 0.45

The compressive strength development in the concrete specimens with EAFD content varying from 5 to 15%, as replacement of cement, is depicted in Figure 4.29. The compressive strength increased with age in all the concrete specimens. The compressive strength of EAFD cement concrete specimens was less than that of the plain cement concrete. However, the compressive strength of 10 and 15% EAFD cement concrete was

almost similar while 5% EAFD cement concrete was more than 0% EAFD cement concrete after 90 days of curing. After 90 days of curing, the compressive strength of 0, 5, 10 and 15% EAFD cement concrete specimens was 59.8, 61.1, 52.0 and 50.5 MPa, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

For the 50 and 60% EAFD cement concrete, the EAFD was thermally treated as there was significant delay in the setting time, almost 6 days. Hence, the EAFD was heated at a temperature of 900^0C for about 2 hours. The compressive strength development in the concrete specimens with thermally treated EAFD content of 50 and 60%, as replacement of cement, is depicted in Figure 4.30. The compressive strength increased with age in all the concrete specimens. The compressive strength development in 0% EAFD was more than that of 50 and 60% thermally treated EAFD. However, the compressive strength of 50% EAFD cement concrete specimens was more than that of specimens with 60% EAFD. After 90 days of curing, the compressive strength of 0, 50 and 60% EAFD cement concrete specimens was 29.7, 23.9 and 18.5 MPa, respectively.

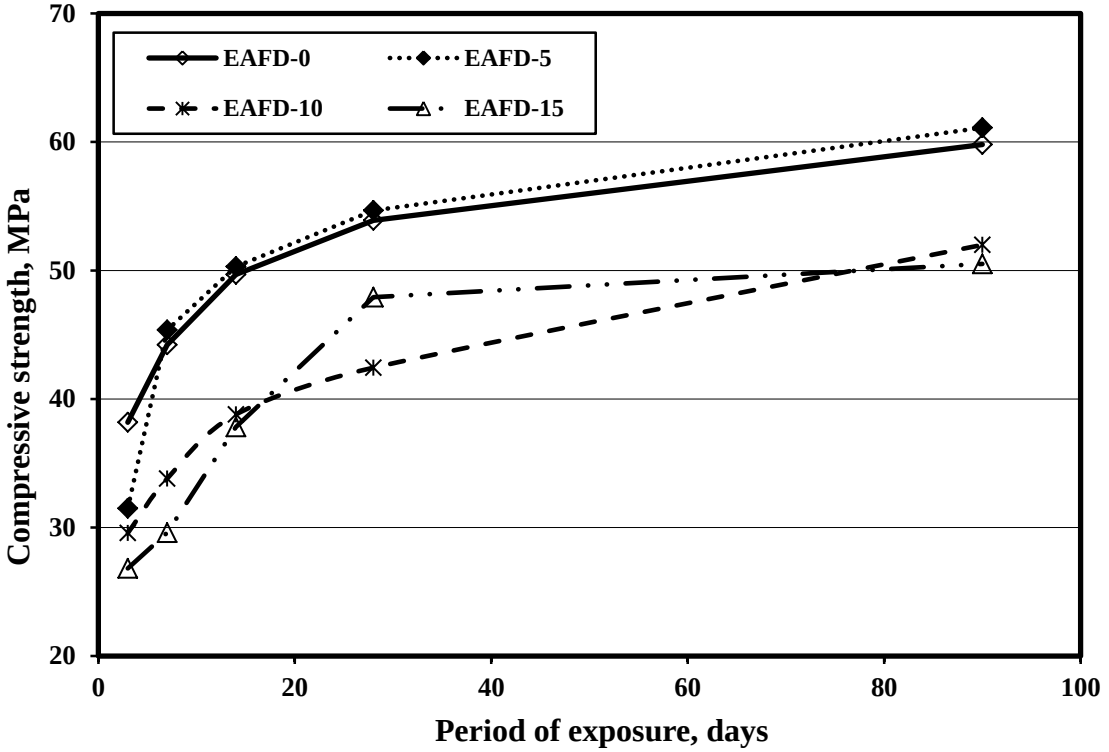

Figure 4-29: Compressive Strength of Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.45.

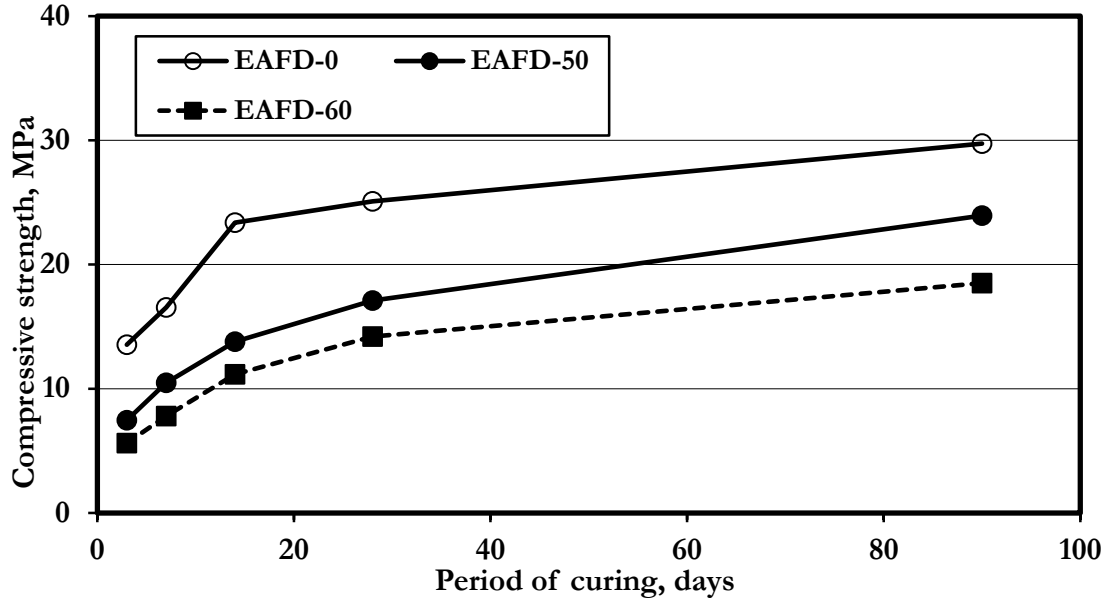

Figure 4-30: Compressive Strength of Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

Referring the data in Figures 4.29 and 4.30 and in Table 4.1, the results indicate that the 5% EAFD cement concrete can be utilized as high strength concrete since the strength was more than 50 MPa, while 10 and 15% EAFD cement concrete as medium strength since the strength was more than 30 MPa. Concretes with 50% and 60% thermally treated EAFD can be utilized as very low strength, respectively.

4.3.2 Water Absorption

Concrete Specimens with a w/cm Ratio of 0.45

The water absorption of the concrete specimens with EAFD varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.31. The water absorption of concrete specimens with 5, 10, and 15% EAFD content was more than that of plain cement concrete, and water absorption of concrete specimens increased with an increase in the quantity of EAFD. After 28 days of curing, the water absorption of 0, 5, 10, and 15% EAFD cement concrete was 4.1, 4.3, 4.6, and 4.9%, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The water absorption of the concrete specimens with 50 and 60% EAFD, as replacement of cement, is depicted in Figure 4.32. The water absorption of 50 and 60% EAFD was marginally more than that of 0% EAFD. However, the water absorption of 50 and 60% EAFD cement concrete specimens was almost the same. After 28 days of curing, the water absorption of 0, 50 and 60% EAFD cement concrete specimens was 5.3, 5.5, and 5.5%, respectively.

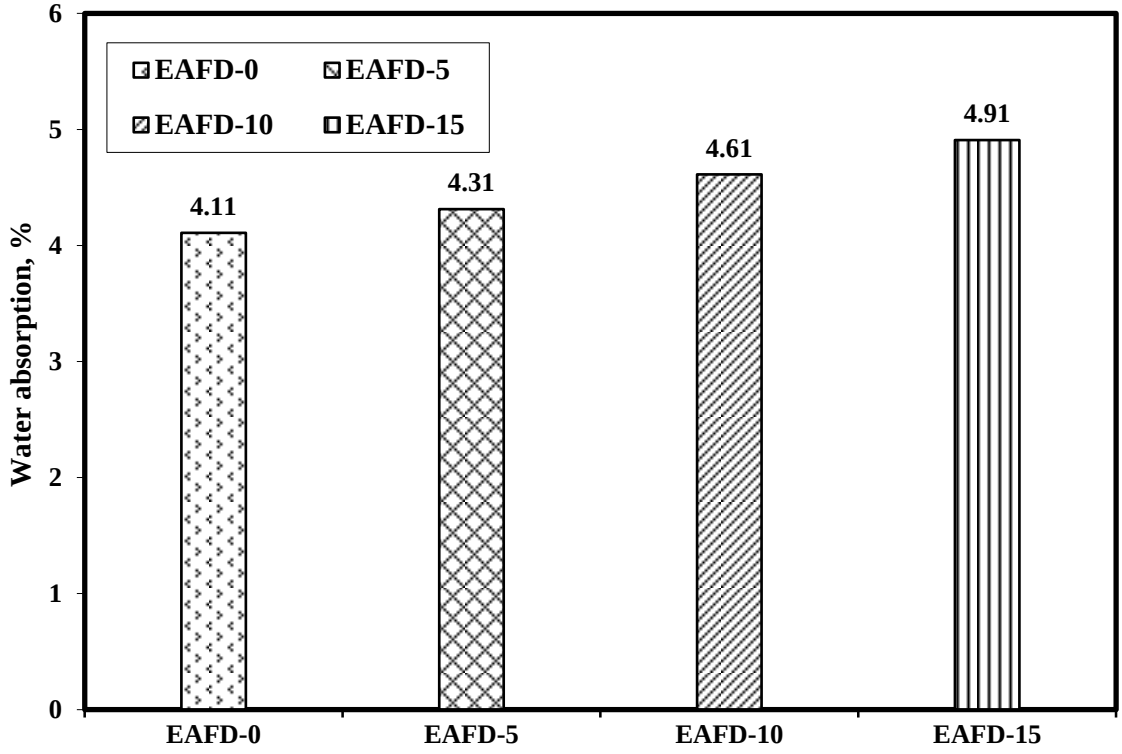

Figure 4-31: Water Absorption of Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.45.

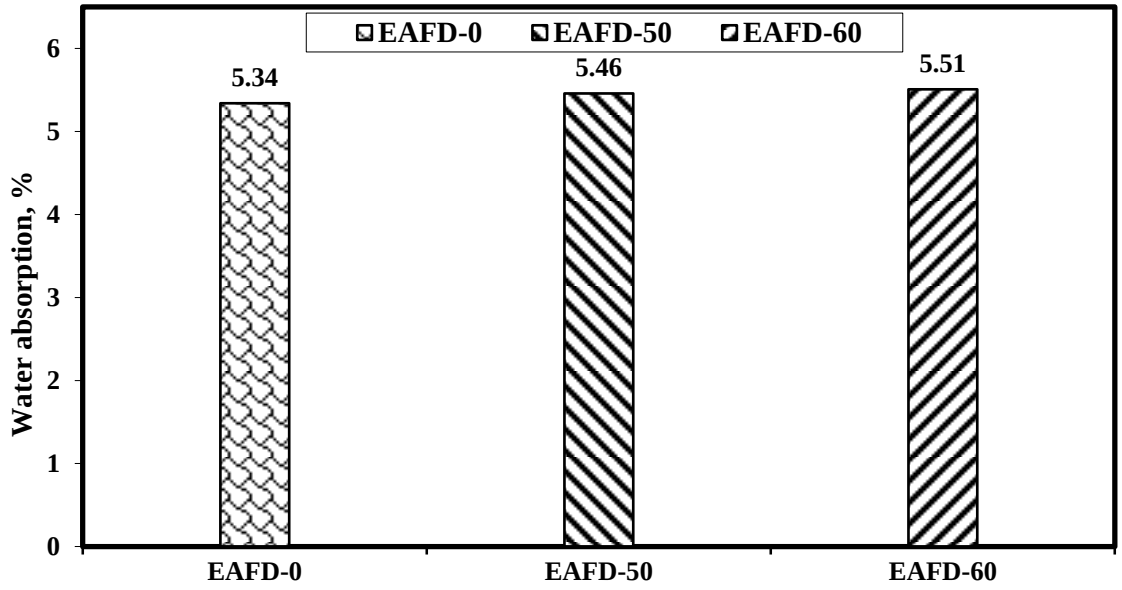

Figure 4-32: Water Absorption of Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

4.3.3 Corrosion Potentials

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion potentials on steel in the concrete specimens prepared with EAFD content varying from 0 to 15%, as replacement of cement, are depicted in Figure 4.33. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the concrete specimens. Corrosion potentials on steel in the 15% EAFD cement concrete specimens were more than that on plain cement concrete specimens thereby resulting in reduction of the time of initiation of corrosion. Corrosion potentials on steel in 5 and 10% EAFD cement concrete specimens are almost same and less than plain cement concrete. The time to initiation of corrosion was about 58, 19, 23, and 9 days for 0, 5, 10, and 15% EAFD cement concrete, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The corrosion potentials on steel in the concrete specimens prepared with 50 and 60% EAFD, as replacement of cement, are depicted in Figure 4.34. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the concrete specimens. The corrosion potentials for 50% EAFD cement concrete were slightly more than 60% EAFD cement concrete. However, the corrosion potentials in both the concretes were more than threshold value of -270 mV SCE from the beginning of exposure.

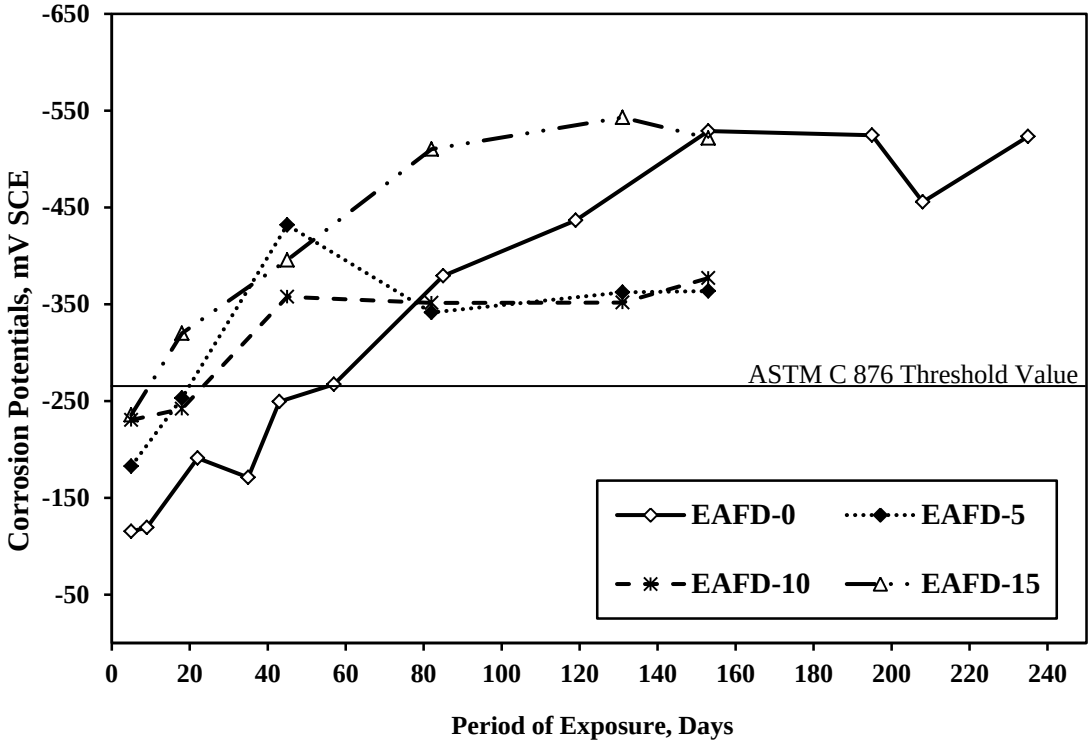

Figure 4-33: Corrosion Potentials on Steel in the Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.45.

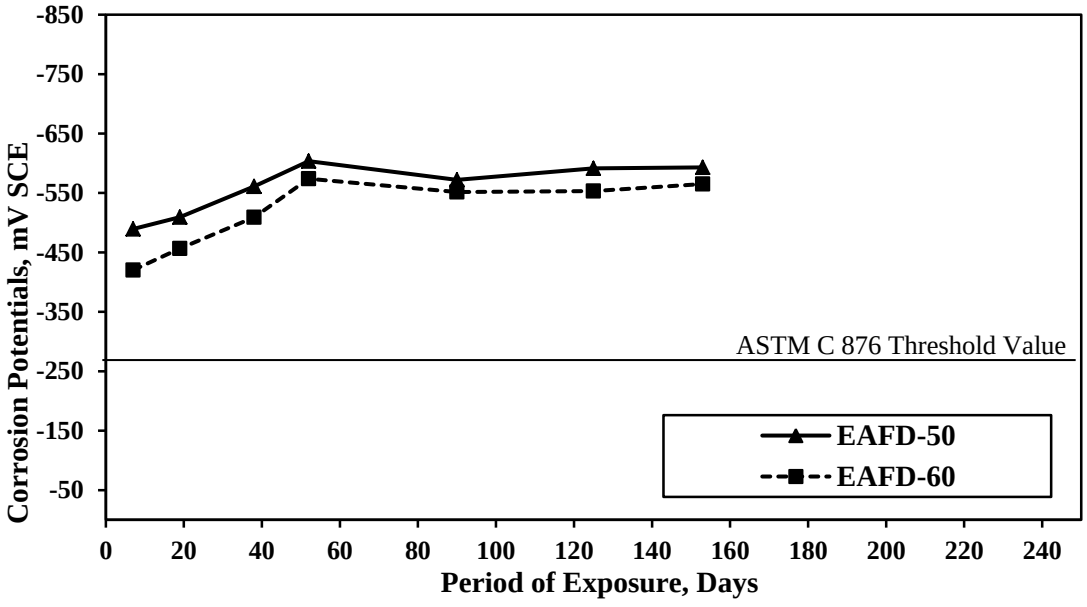

Figure 4-34: Corrosion Potentials on steel in the Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

4.3.4 Corrosion Current Density

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion current density (I_{corr}) on steel in the concrete specimens prepared with EAFD content varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.35. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. The I_{corr} increased with addition of EAFD at early ages. The I_{corr} for 10 and 15% EAFD cement concrete specimens was almost same. The I_{corr} for 5% EAFD was less than that of 10 and 15% EAFD cement concrete. However, after 150 days, the I_{corr} was 0.64, 0.52, 0.61 and 0.63 $\mu A/cm^2$, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The I_{corr} on steel in the concrete specimens prepared with 50 and 60% EAFD, as replacement of cement, is depicted in Figure 4.36. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. I_{corr} for 50% EAFD cement concrete was more than that of 60% EAFD cement concrete. After 150 days, the I_{corr} was 0.66 and 0.63 $\mu A/cm^2$, respectively.

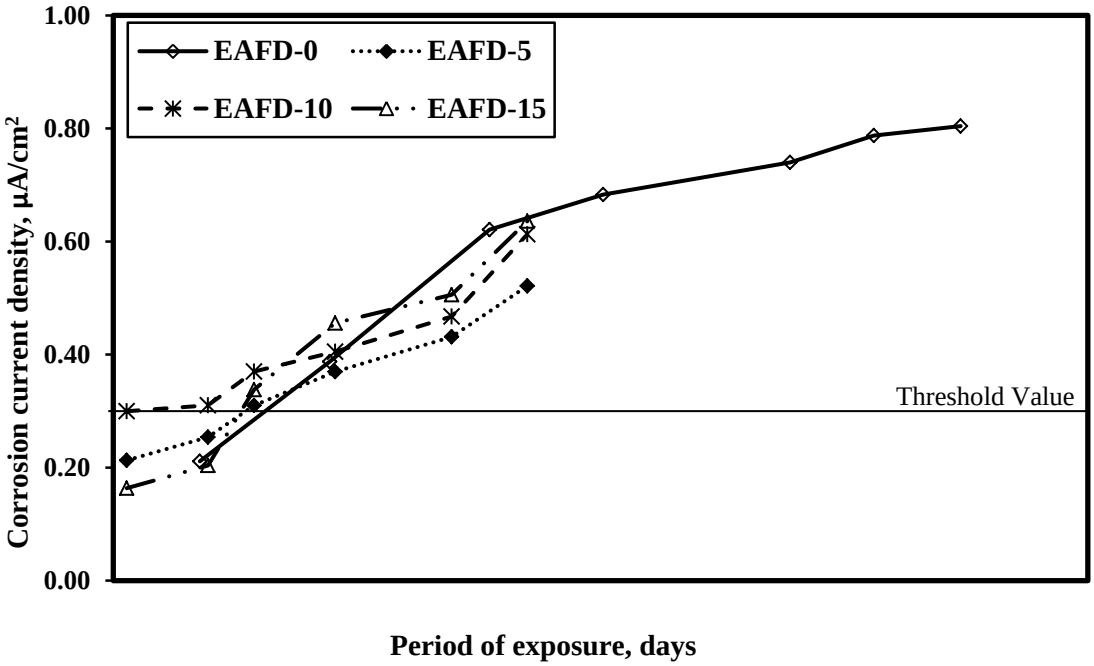

Figure 4-35: Corrosion Current Density on Steel in the Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.55.

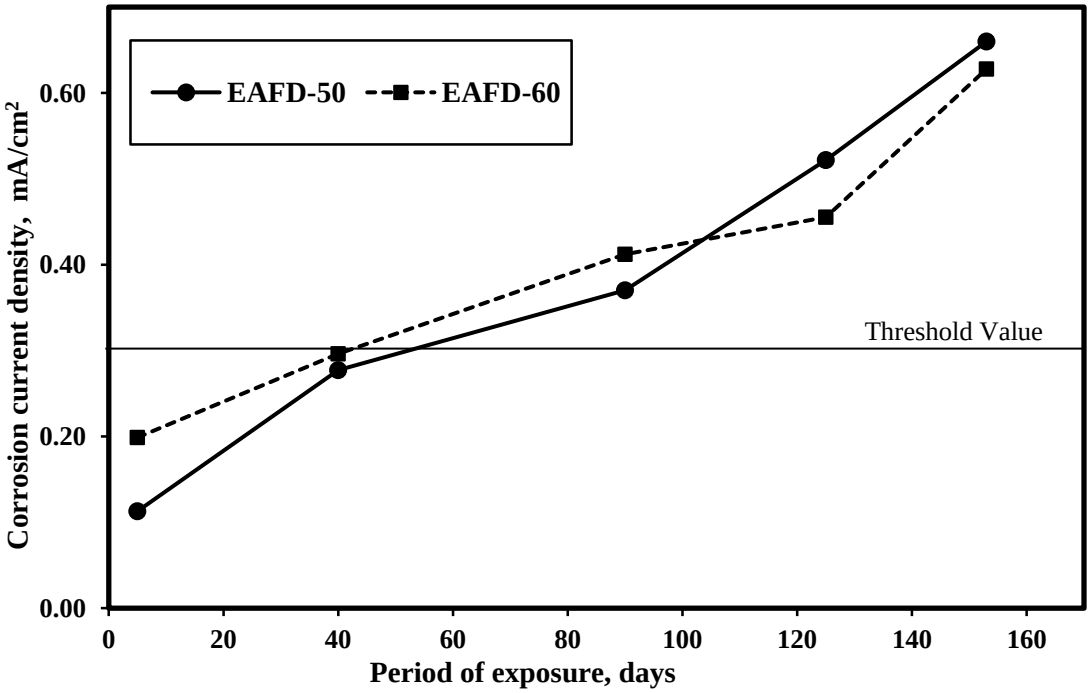

Figure 4-36: Corrosion Current Density on Steel in the Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

4.3.5 Electrical Resistivity

Concrete Specimens with a w/cm Ratio of 0.45

The electrical resistivity of concrete specimens with EAFD content varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.37. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. The electrical resistivity was almost same for 5 to 15% EAFD cement concrete and was more than that for plain cement concrete specimens. The electrical resistivity at 3% moisture content for 0, 5, 10 and 15% EAFD cement concrete was 34.2, 38.8, 39.2, and 39.2 kΩ-cm, respectively. With regard to risk of reinforcement corrosion, all the specimens are in the moderate range.

Concrete Specimens with a w/cm Ratio of 0.55

The electrical resistivity of concrete specimens prepared with 50 and 60% EAFD, as replacement of cement, is depicted in Figure 4.38. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. The electrical resistivity at 3% moisture content for 50 and 60% EAFD cement concrete was 32.0 and 26.8 kΩ-cm, respectively. With these results, it is observed that electrical resistivity decreased with increase in EAFD content, and with regard to risk of reinforcement corrosion the specimens are in the moderate range.

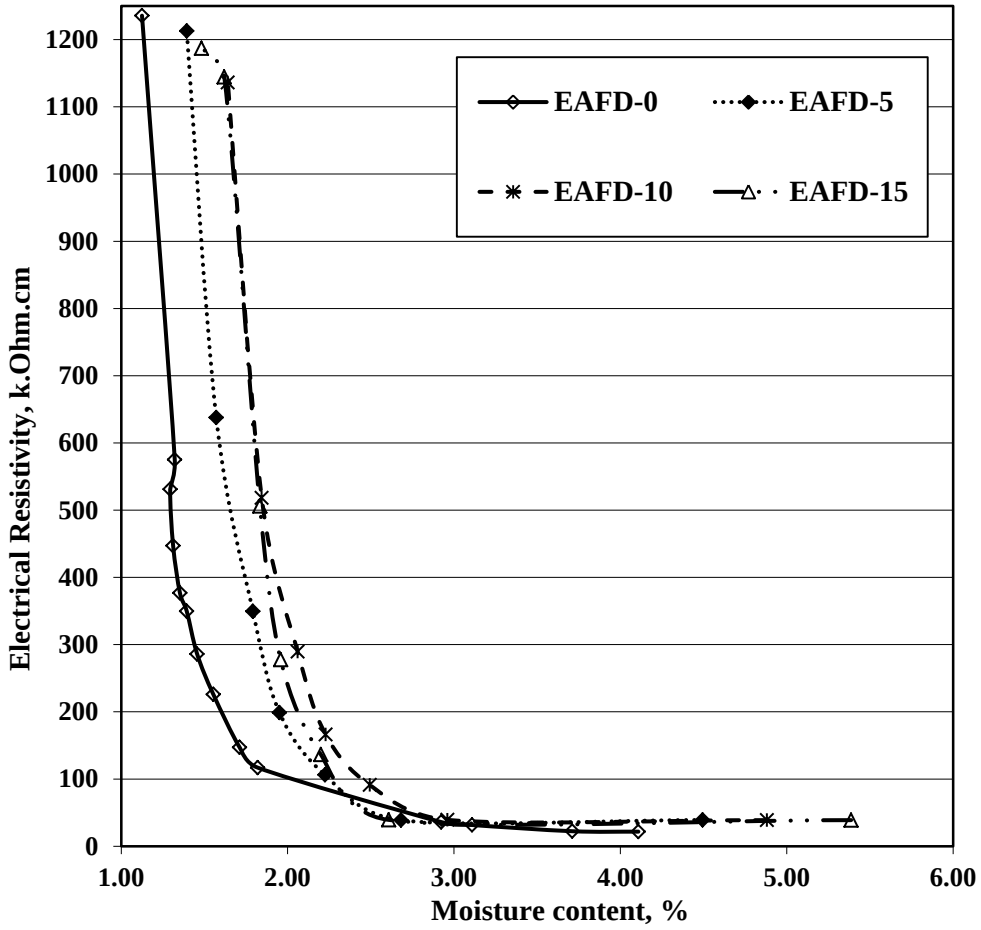

Figure 4-37: Electrical Resistivity of Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.45.

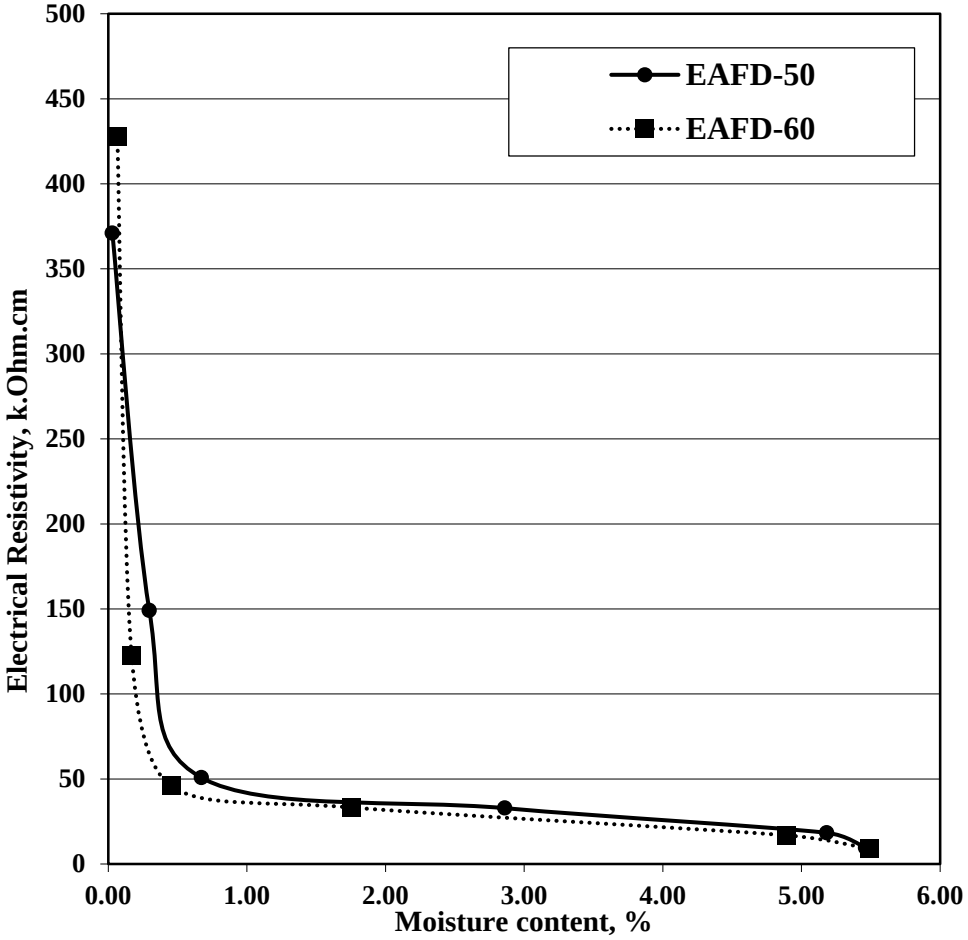

Figure 4-38: Electrical Resistivity of Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

4.3.6 Thermal Conductivity

Concrete Specimens with a w/cm Ratio of 0.45

The thermal conductivity of the concrete specimens with EAFD varying from 0 to 15%, as replacement of cement, is depicted in Figure 4.39. The thermal conductivity increased with an increase in the EAFD content. However, the increase in thermal conductivity with an increase in the quantity of EAFD is marginal. After 28 days of curing, the thermal

conductivity of 0, 5, 10, and 15% EAFD cement concrete was 2.4, 2.4, 2.5, and 2.5 W/mk, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The thermal conductivity of the concrete specimens with 50 and 60% EAFD, as replacement of cement, is depicted in Figure 4.40. The thermal conductivity slightly decreased with an increase in the quantity of EAFD in concrete specimens. After 28 days of curing, the thermal conductivity of 50 and 60% EAFD cement concrete was 2.6, and 2.3 W/mk, respectively.

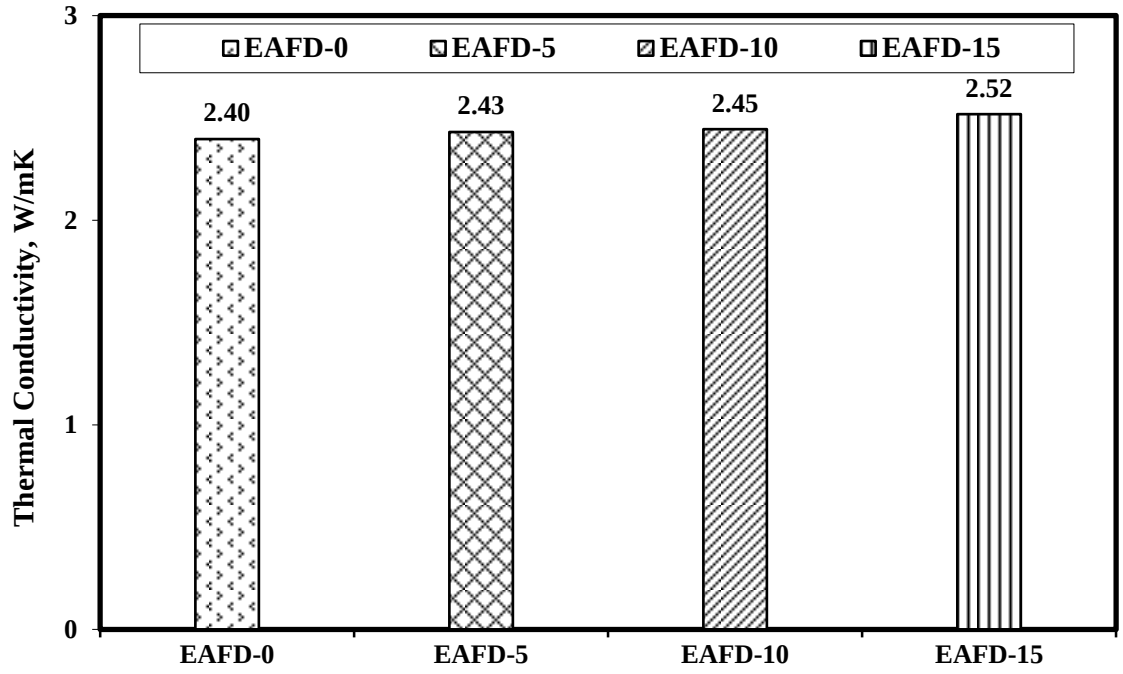

Figure 4-39: Thermal Conductivity of Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.55.

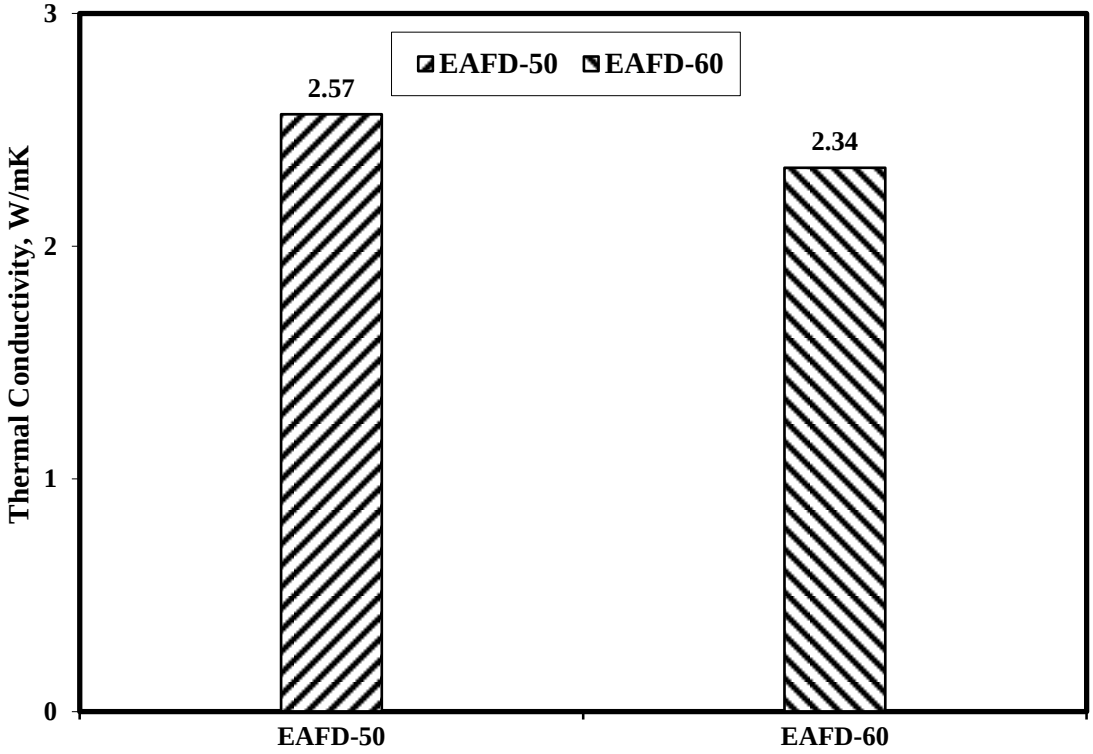

Figure 4-40: Electrical Resistivity of Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

4.3.7 Drying Shrinkage

Concrete Specimens with a w/cm Ratio of 0.45

The drying shrinkage in the concrete specimens prepared with EAFD content varying from 0 to 15%, is depicted in Figure 4.41. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, stabilizing with time and remaining almost unchanged thereafter. The drying shrinkage for 15% EAFD content was more than that for 0, 5 and 10% EAFD concrete. The drying shrinkage for 5% EAFD cement concrete was less than that for plain cement concrete specimens after 140 days. At this age the drying shrinkage

of specimens with 0, 5, 10 and 15% EAFD cement concrete was 532, 480, 544, and 600 microns, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The drying shrinkage of concrete specimens with 50 and 60% EAFD is depicted in Figure 4.42. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, stabilizing with time and remaining almost unchanged thereafter. The drying shrinkage of 60% EAFD cement concrete was more than that of 50% EAFD cement concrete. After 140 days, the drying shrinkage of specimens with 50 and 60% EAFD cement concrete was 624 and 786 microns, respectively.

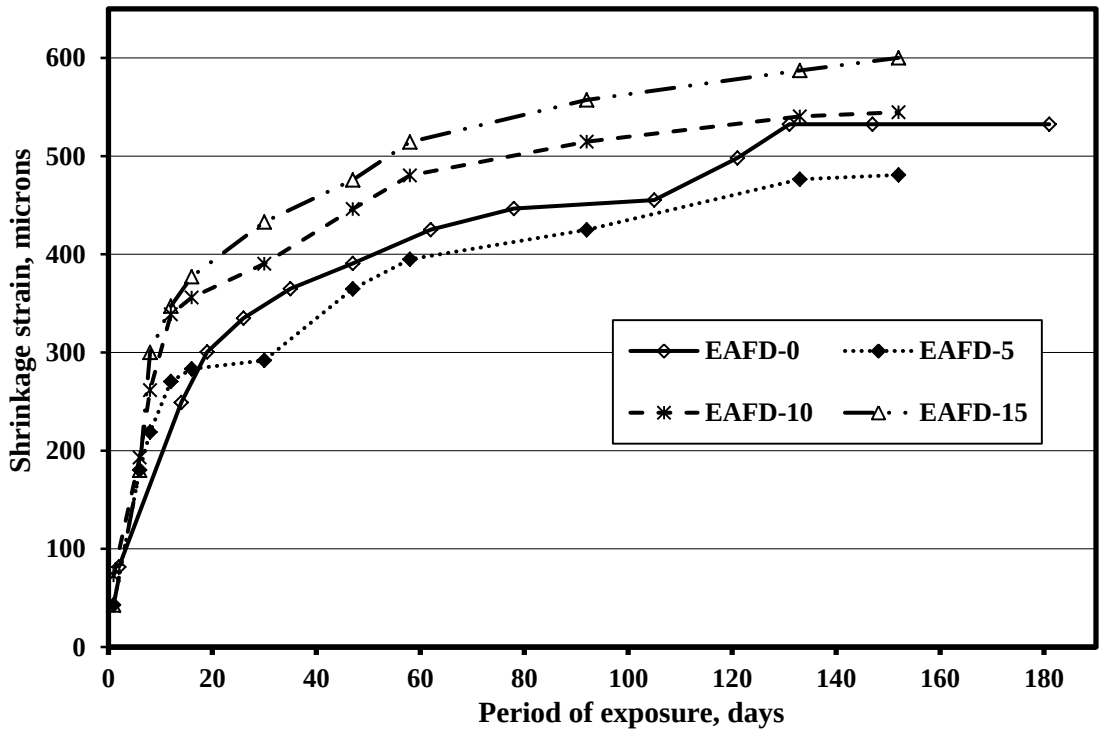

Figure 4-41: Drying Shrinkage Strain in Concrete Specimens Prepared with Different Percentages of EAFD and a w/cm Ratio of 0.45.

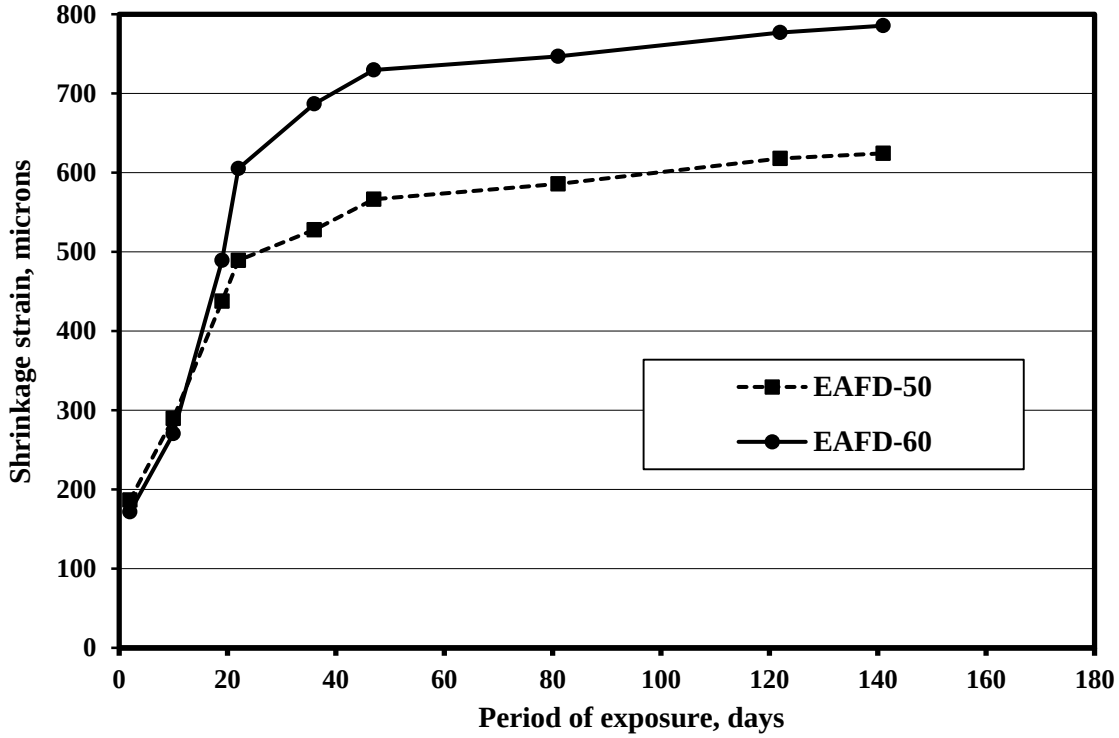

Figure 4-42: Drying Shrinkage Strain in Concrete Specimens Prepared with 50 and 60% EAFD and a w/cm Ratio of 0.55.

4.3.8 Setting Time

The normal consistency and setting time of normal (no thermal treatment) EAFD cement (varying from 0 to 60%, as replacement), is depicted in Table 4.1. The normal consistency and setting time of thermally treated EAFD content with 50 and 60% as replacement of cement is depicted in Table 4.2. Setting time indicates the rate at which the hydration reactions are taking place.

From the data in Table 4.1, it can be noted both the initial and final setting time increase with an increase in normal EAFD content. As EAFD is rich in zinc and lead, when EAFD reacts with water, a membrane is formed around cement particles with the precipitation of calcium

hydroxyzincate that will prevent water and ion transport needed for cement hydration, this leads to retardation of cement hydration [84].

Table 4-2: Initial and Final Setting Time of Cement Blended with Normal EAFD Cement Concrete.

Mix	Consistency, % by weight of Mix	Time of setting, Min	
		Initial	Final
OPC	25.7	200	265
EAFD (5%)	26.6	325	415
EAFD (10%)	24.8	525	615
EAFD (15%)	31.6	776	1123
EAFD (50%)	28	1230	1470
EAFD (60%)	27.2	1423	1672

Table 4-3: Initial and final setting time of Cement Blended with Thermally Treated EAFD.

Mix	Consistency, % by weight of Mix	Time of setting, Min	
		Initial	Final
EAFD (50%)	38.6	325	415
EAFD (60%)	35.3	525	615

From Table 4.2, the desperate change in setting time of 50 and 60% thermally treated EAFD cement concrete was observed. EAFD was heated for 2 hours at temperature 900^{0}C. Initial setting time for 50% EAFD decreased from 1230 min to 325 min, and for 60% from 1423 min to 525 min. Final setting time for 50% EAFD decreased from 1470 min to 415 min, and for 60% from 1672 min to 615 min. It can be clearly observed that the property of EAFD enhanced with heating. Hence, the problem of retarding can be overcome by heat treatment.

4.3.9 Leachability of Heavy Metals

Leaching test was performed on concrete specimens with 5, 10, 15, 50 and 60% EAFD according to TCLP method EPA 1311. The aim was to measure the concentration of any leachable hazardous substance that is harmful to environment. The results are depicted in Table 4.3, with the limits recommended by US EPA in mg/kg (i.e., ppm). The leachate samples were analysed for concentration of heavy toxic metals.

Table 4-4: Concentrations of TCLP Elements in EAFD Cement Concrete Specimens.

TCLP-Elements	EAFD 5%	EAFD 10%	EAFD 15%	EAFD 50%	EAFD 60%	TCLP limits
	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg
Ag	<0.010	<0.010	<0.010	<0.010	<0.010	5.0
Al	<0.003	<0.003	<0.003	<0.003	<0.003	--
As	<0.006	<0.006	<0.006	<0.006	<0.006	5.0
Ba	0.117	0.123	0.119	0.114	0.103	100.0
Cd	<0.001	<0.001	<0.001	<0.001	<0.001	1.0
Co	<0.001	<0.001	<0.001	<0.001	<0.001	--
Cr	0.271	0.308	0.359	1.38	1.86	5.0
Fe	<0.050	<0.050	<0.050	<0.050	<0.050	--
Hg	<0.050	<0.050	<0.050	<0.050	<0.050	0.1
Mn	<0.002	<0.002	<0.002	0.020	0.010	--
Ni	<0.002	<0.002	<0.002	0.010	0.013	25
Pb	<0.010	<0.010	<0.010	<0.010	<0.010	5.0
V	0.091	0.074	0.063	0.069	0.064	25
Zn	<0.003	<0.003	<0.003	0.712	0.650	250

The concentration of barium, chromium and zinc are found to be slightly higher when compared to other leached metals. However, the concentration values of all heavy metals tested in the leachate samples are well within the allowable limits. Hence, the EAFD cement concrete specimens are classified as non-hazardous.

4.4 Properties of OA Cement Concrete

4.4.1 Compressive Strength Development

Concrete Specimens with a w/cm Ratio of 0.45

The compressive strength development of concrete specimens with OA content of 5 and 10%, as replacement of cement, is depicted in Figure 4.43. The compressive strength increased with age in both concrete specimens. The initial compressive strength of 5% OA was more than 10% OA cement concrete. However, it was less than 0% OA cement concrete. Whereas, the compressive strength development of 5 and 10% OA cement concrete was almost similar and more than control specimen at an age of 90 days. After 90 days of curing, the compressive strength of 0, 5, and 10% OA cement concrete specimens was 59.8, 64 and 65.2 MPa, respectively.

Referring the data in Figure 4.43 and in Table 4.1, the results indicate that 5 and 10% OA can be utilized as high and medium strength concrete since the strength was more than 50 and 30 MPa, respectively.

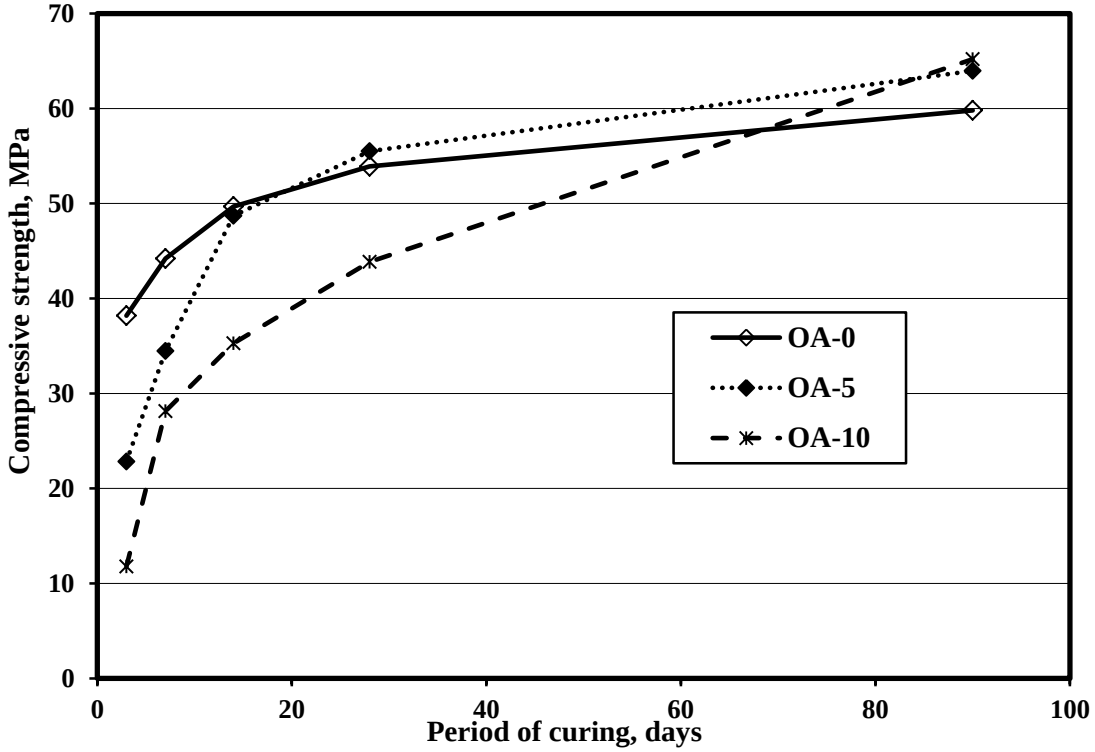

Figure 4-43: Compressive Strength of Concrete Specimens Prepared with Different Percentages of OA.

4.4.2 Water Absorption

The water absorption in the concrete specimens with OA varying from 0 to 10%, as replacement of cement, is depicted in Figure 4.44. The water absorption increased with an increase in the quantity of OA. The water absorption of 10% OA cement concrete was much higher than that of 0 and 5% OA content. After 28 days of curing, the water absorption of 0, 5, and 10% OA was 4.1, 4.4, and 5.2%, respectively.

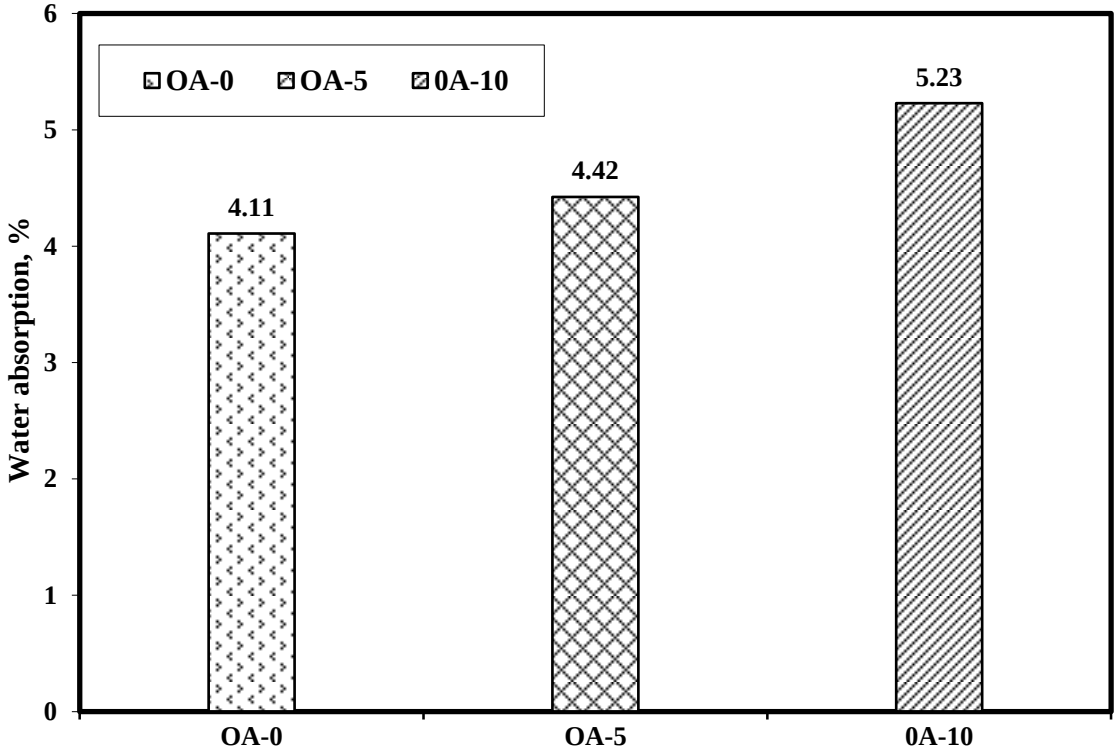

Figure 4-44: Water Absorption of Concrete Specimens Prepared with Different Percentages of OA.

4.4.3 Corrosion Potentials

The corrosion potentials on steel in the concrete specimens prepared using OA content varying from 0 to 10%, as replacement of cement, are depicted in Figure 4.45. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the concrete specimens. The corrosion potentials in the 5 and 10% OA cement concrete was almost same and more than that in the plain cement concrete. The time to initiation of corrosion was about 58, and 8 days for 0 and 5%, respectively, however, for 10% OA it was less than 5 days.

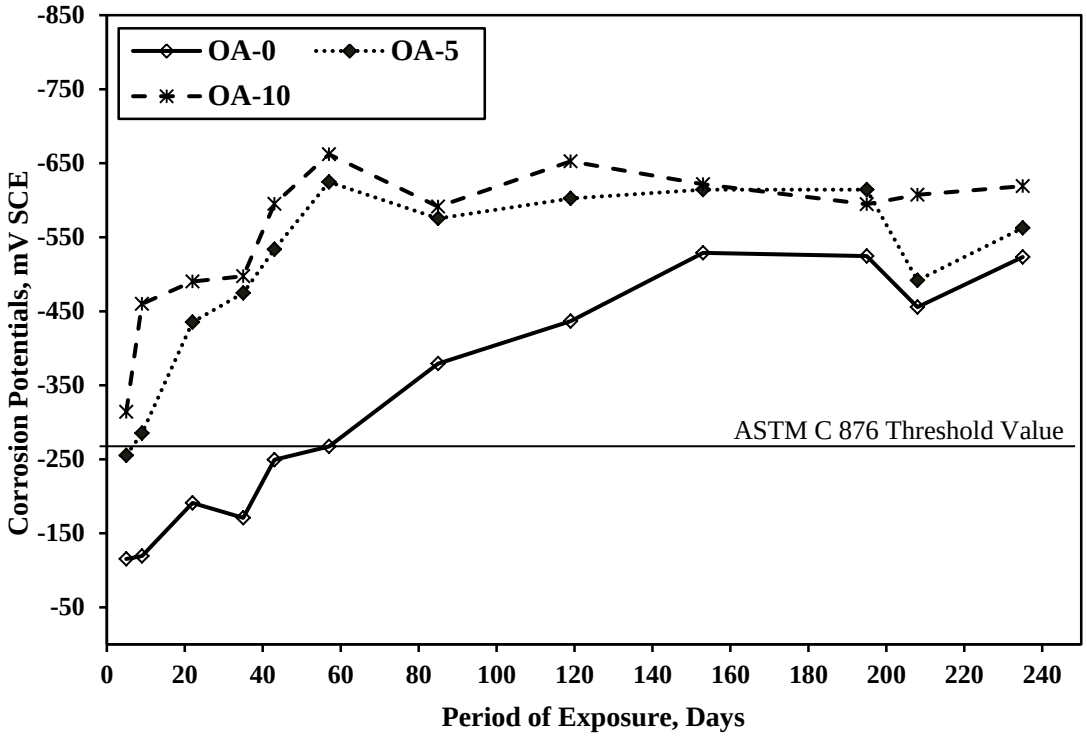

Figure 4-45: Corrosion Potentials on Steel in the Concrete Specimens Prepared with Different Percentages of OA.

4.4.4 Corrosion Current Density

The corrosion current density (I_{corr}) on steel in the concrete specimens prepared using OA content varying from 0 to 10%, as replacement of cement, is depicted in Figure 4.46. Initially the I_{corr} on steel in all concrete specimes was almost the same at 0.23 $\mu A/cm^2$. The I_{corr} increased with the period of exposure to the chloride solution in all the concrete specimens. I_{corr} of 5 and 10% OA cement concretes was less than that of the plain cement concrete (0% OA). However, the I_{corr} of 10% OA cement concrete was slightly more than that of 5% OA cement concrete. However, after 250 days, the I_{corr} was 0.60, 0.58 and 0.74 $\mu A/cm^2$ for 0, 5 and 10% OA cement concrete, respectively.

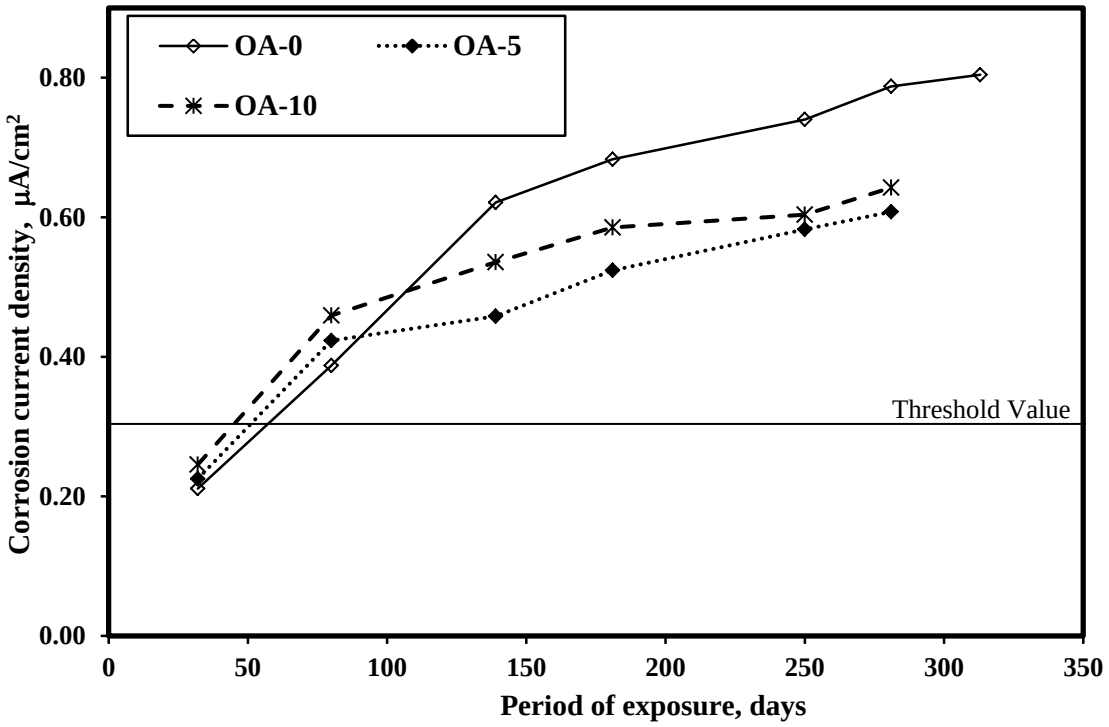

Figure 4-46: Corrosion Current Density on Steel in the Concrete Specimens Prepared with Different Percentages of OA

4.4.5 Electrical Resistivity

The electrical resistivity of concrete specimens prepared with OA content varying from 0 to 10%, as replacement of cement, is depicted in Figure 4.47. The electrical resistivity decreased with an increase in moisture content in all the concrete specimens. The electrical resistivity for the 5 and 10% OA cement concretes was more than that of plain cement concrete. The electrical resistivity at 3% moisture content for 0, 5, and 10% OA was 34.2, 103.424, and 120.1 kΩ-cm, respectively. With regard to risk of reinforcement corrosion, summarised in Table 3.8, the specimens with 5 and 10% OA are in range of low corrosion current intensity and those with 0% OA are of moderate corrosion current intensity.

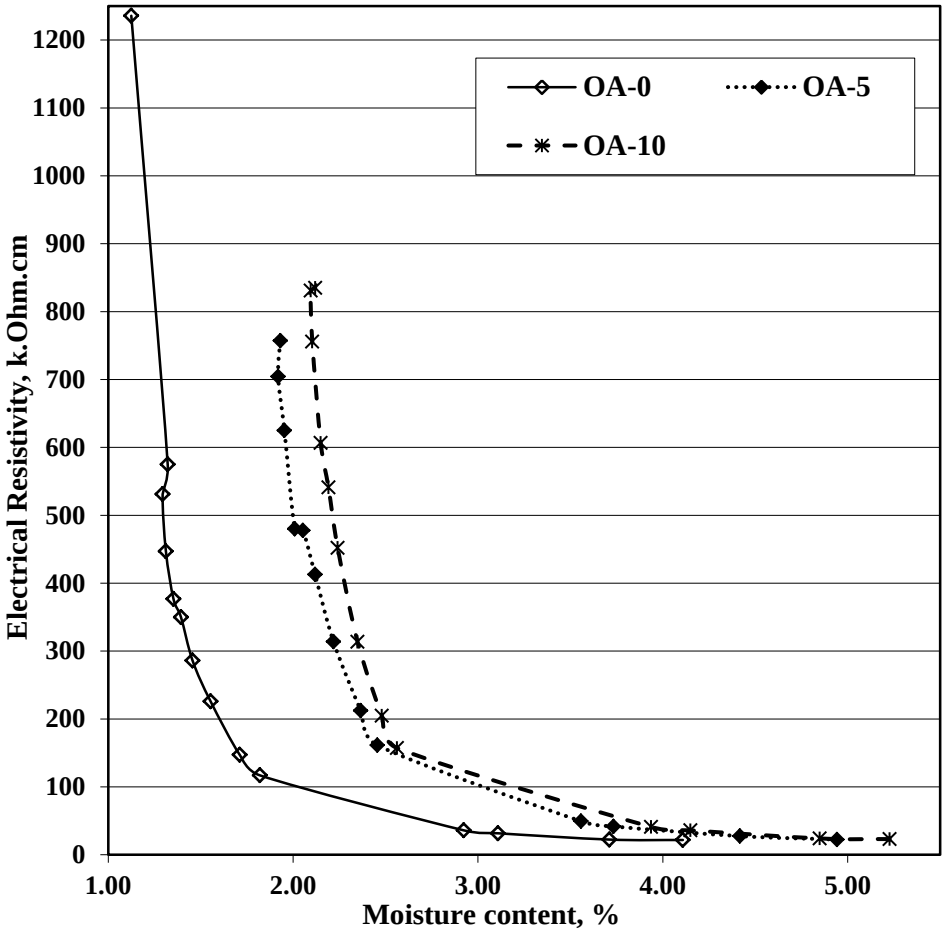

Figure 4-47: Electrical Resistivity of Concrete Specimens Prepared with Different Percentages of OA.

4.4.6 Thermal Conductivity

The thermal conductivity of the concrete specimens with OA varying from 0 to 10%, as replacement of cement, is depicted in Figure 4.48. Thermal conductivity increased up to 5% OA content and then reduced with an increase in the OA content. After 28 days of curing, the thermal conductivity of 0, 5, and 10% OA was 2.4, 2.51, and 2.37 W/mk, respectively.

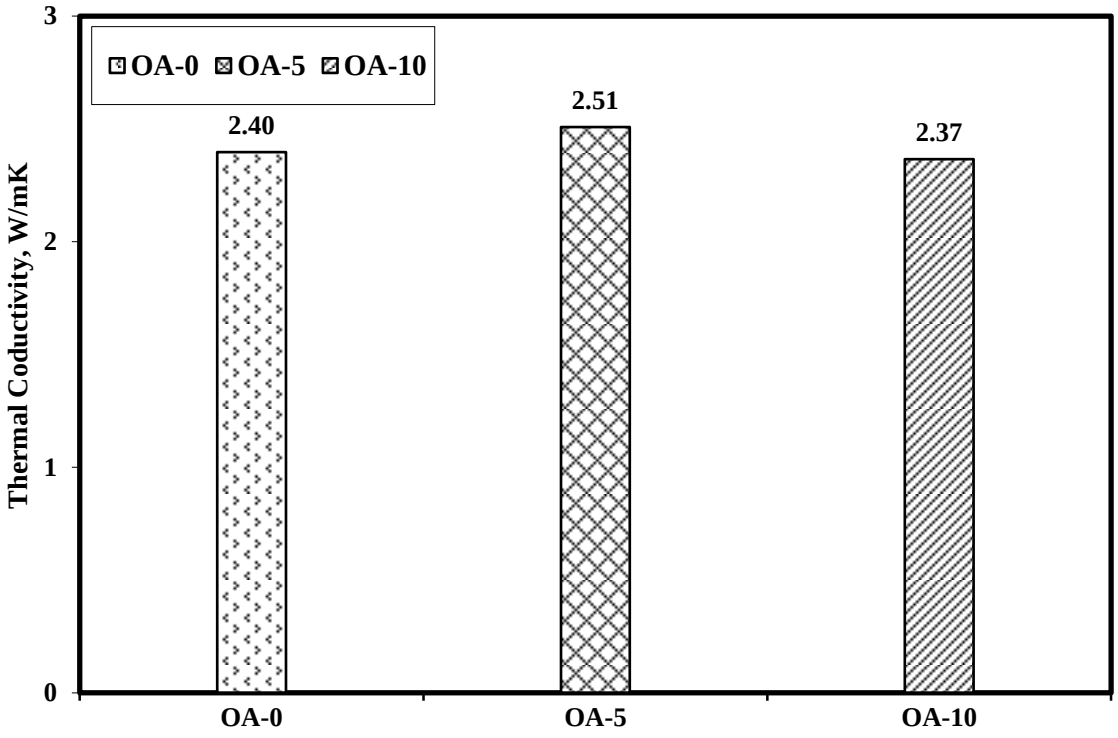

Figure 4-48: Thermal Conductivity of Concrete Specimens Prepared with Different Percentages of OA.

4.4.7 Drying Shrinkage

The drying shrinkage in the 0, 5 and 10% OA cement concrete specimens is depicted in Figure 4.49. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially and then stabilized with time. The drying shrinkage in the specimen with 5 and 10% OA was slightly less than that in plain concrete specimen. Further, the drying shrinkage of 5 and 10% OA cement concrete was almost similar. After 180 days, the drying shrinkage of specimens with 0, 5 and 10% OA cement concrete was 532, 496, and 510 microns, respectively.

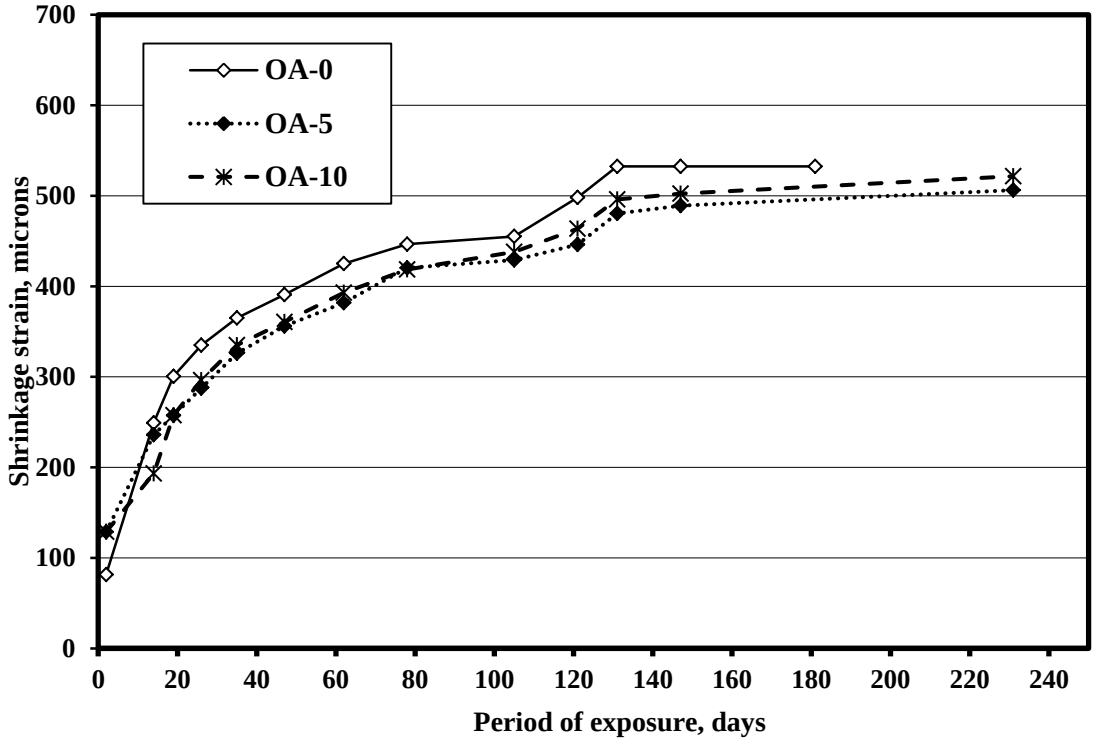

Figure 4-49: Drying Shrinkage Strain in Concrete Specimens Prepared with Different Percentages of OA.

4.4.8 Leachability of Heavy Metals

Leaching test was performed on concrete specimens with 5 and 10% OA according to TCLP method EPA 1311. The aim was to measure the concentration of any leachable hazardous substance that is harmful to environment. The results are depicted in Table 4.5, with limits recommended by US EPA. The leachate samples were analysed for concentration of heavy toxic metals.

The concentration of chromium (i.e., 0.227 and 0.191 mg/kg for 5 and 10% OA) and vanadium (i.e., 2.27 and 4.70 mg/kg for 5 and 10% OA) for are found little higher when compared to other heavy metals. However, the concentration values of all heavy metals

tested in the leachate samples are well within the allowable limits. Hence, the OA cement concrete specimens are classified as non-hazardous.

Table 4-5: Concentrations of TCLP Elements in OA Cement Concrete Specimens.

TCLP-Elements	**Oil ash 5%**	**Oil ash 10%**	**TCLP limits**
	mg/kg	**mg/kg**	**mg/kg**
Ag	<0.010	<0.010	5.0
Al	<0.003	<0.003	--
As	<0.006	<0.006	5.0
Ba	0.089	0.062	100.0
Cd	<0.001	<0.001	1.0
Co	<0.001	<0.001	--
Cr	0.227	0.191	5.0
Fe	<0.050	<0.050	--
Hg	<0.050	<0.050	0.1
Mn	<0.002	<0.002	--
Ni	0.032	<0.002	25
Pb	<0.010	<0.010	5.0
V	2.27	4.70	25
Zn	<0.003	<0.003	250

4.5 Properties of NP Cement Concrete

4.5.1 Compressive Strength Development

Concrete Specimens with a w/cm Ratio of 0.45

The compressive strength development in the concrete specimens with NP content varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.50. The compressive strength increased with age in all the concrete specimens. The initial compressive strength of 15 and 20% NP cement concrete was almost the same at 24 MPa, while for the 10% NP cement concrete (i.e., 34.6 MPa) it was higher than the 15 and 20% NP cement concrete (i.e., 24 MPa). After 90 days of water curing, the compressive strength development of 10% NP was found to be more than that of plain and other NP mixes. After 90 days of water curing, the compressive strength of 0, 10, 15, and 20% NP cement concrete specimens was 59.8, 69.3, 58.0, and 53.1 MPa, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The compressive strength development in the concrete specimens with 50 and 60% NP, used as replacement of cement, is depicted in Figure 4.51. The compressive strength increased with age in all the concrete specimens. Further, the compressive strength tended to decrease with an increase in the NP content. The compressive strength development of 50% NP content was more than that of 60% NP cement concrete. However, the difference in strength was not significant. After 90 days of curing, the compressive strength of 0, 50 and 60% NP cement concrete specimens was 29.7, 27.5 and 24.5 MPa, respectively.

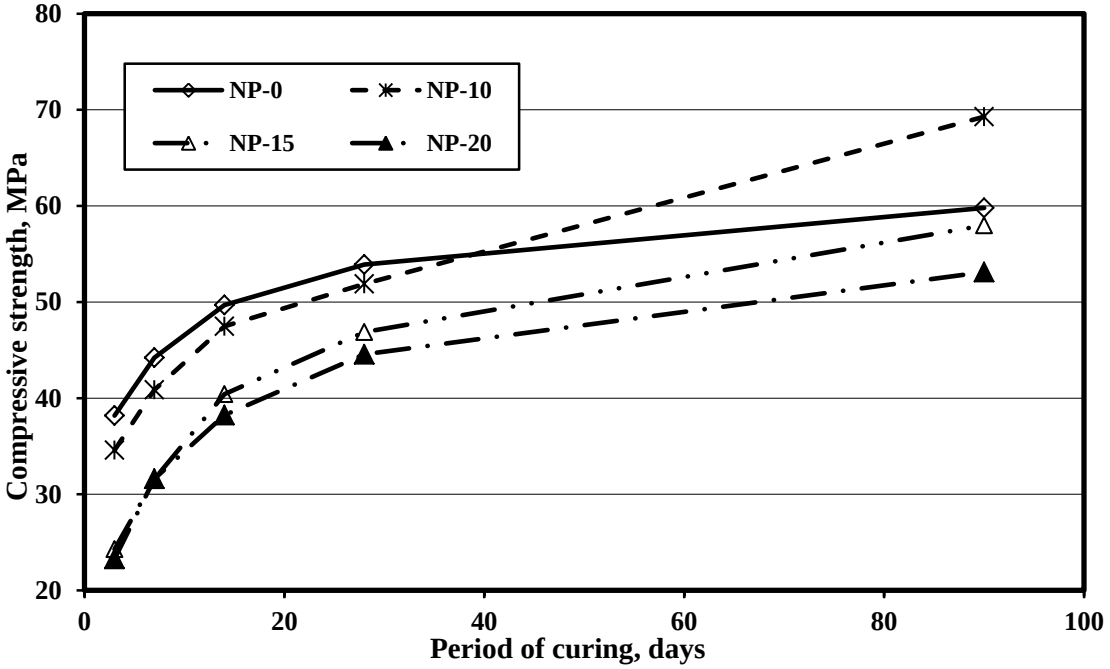

Figure 4-50: Compressive Strength of Concrete Specimens Prepared with Different Percentages of NP and a w/cm Ratio of 0.45.

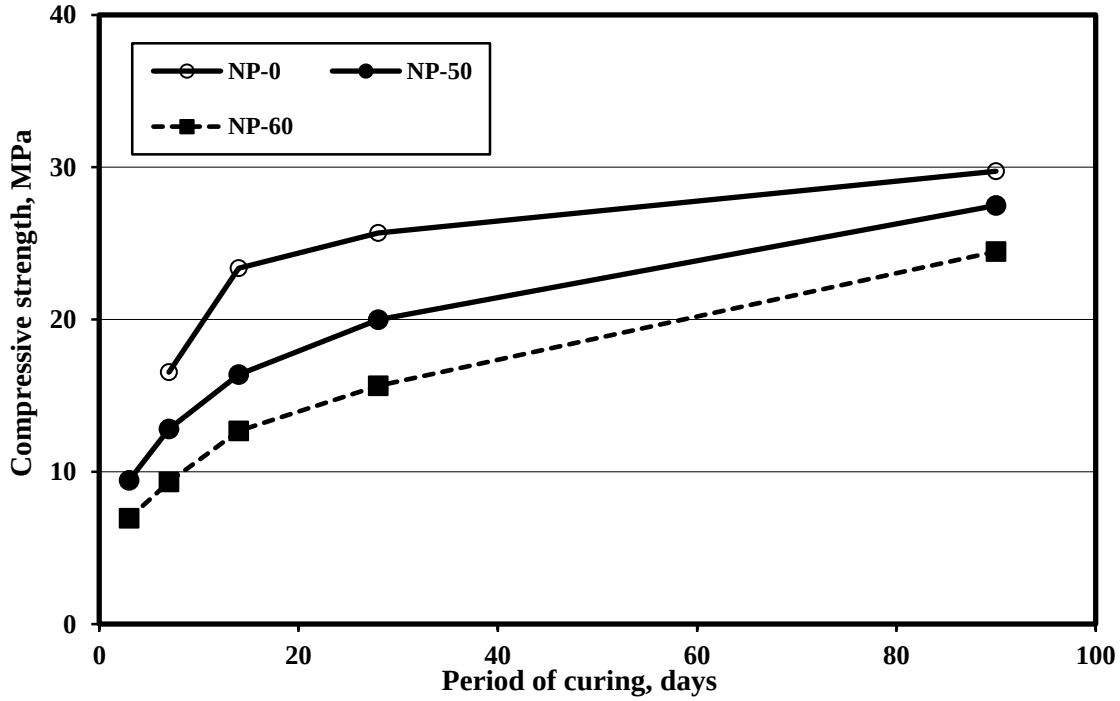

Figure 4- 51: Compressive Strength of Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

Referring the data in Figures 4.50 and 4.51 and Table 4.1, the results indicate that 10% NP can be utilized as high strength concrete since strength was more than 50 MPa, while 15 and 20% as medium strength concrete since the strength was more than 30 MPa. Concretes with 50 and 60% NP can be utilized as medium and low strength concrete, respectively.

4.5.2 Water Absorption

Concrete Specimens with a w/cm Ratio of 0.45

The water absorption in the concrete specimens with NP varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.52. The water absorption increased with an increase in the quantity of NP content. Water absorption of specimens with NP content was more than that of plain cement concrete specimens. After 28 days of curing, the water absorption of 0, 10, 15, and 20% NP cement concrete was 4.1, 4.4, 5, and 5.2%, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The water absorption in the concrete specimens with 50 and 60% NP, used as replacement of cement, is depicted in Figure 4.53. The water absorption increased with an increase in the quantity from 50 to 60% NP. However, the water absorption of 50% NP cement concrete was less than that of 0% NP cement concrete. After 28 days of curing, the water absorption of 0, 50 and 60% NP cement concrete was 5.3, 5.2, and 5.5%, respectively.

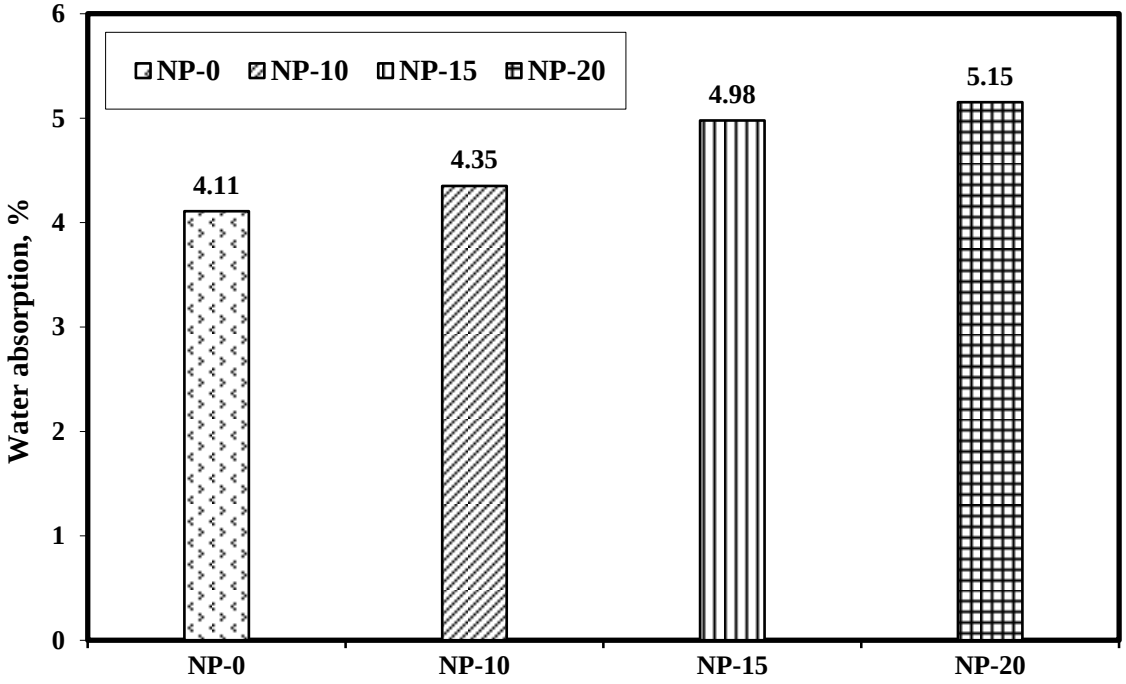

Figure 4-52: Water Absorption of Concrete Specimens Prepared with Different Percentages of NP and a w/cm Ratio of 0.45.

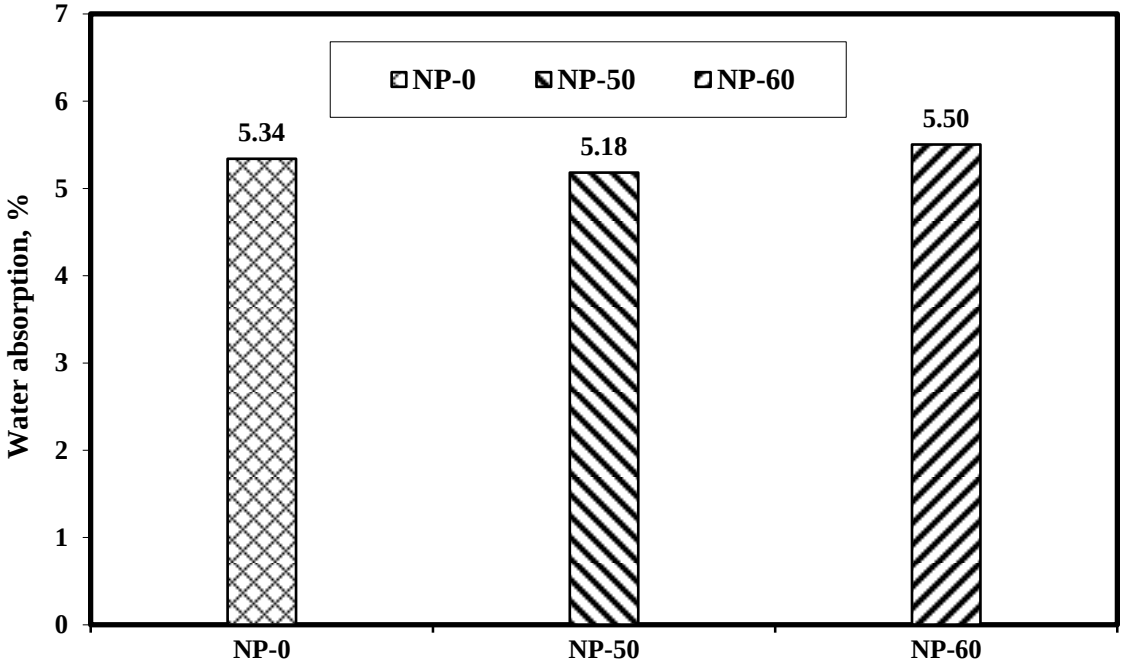

Figure 4-53: Water Absorption of Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

4.5.3 Corrosion Potentials

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion potentials on steel in the concrete specimens prepared with NP varying from 0 to 20%, as replacement of cement, are depicted in Figure 4.54. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the concrete specimens. However, after 230 days, the corrosion potentials were almost the same in all the specimens. The time to initiation of corrosion was about 58, 8, 10, and 65 days for 0, 10, 15, and 20% NP cement concrete. The performance of mix with 20% NP was found to be the best due to its good delay in initiation of corrosion.

Concrete Specimens with a w/cm Ratio of 0.55

The corrosion potentials on steel in the concrete specimens prepared with 50 and 60% NP, as replacement of cement, are depicted in Figure 4.55. The potentials decreased (became more negative) with time of exposure to the chloride solution in all the concrete specimens. Corrosion initiation was noted after about 18 days in the specimen with 50% NP cement concrete. The potentials were more negative than the threshold value from the beginning of exposure in specimens with 60% NP.

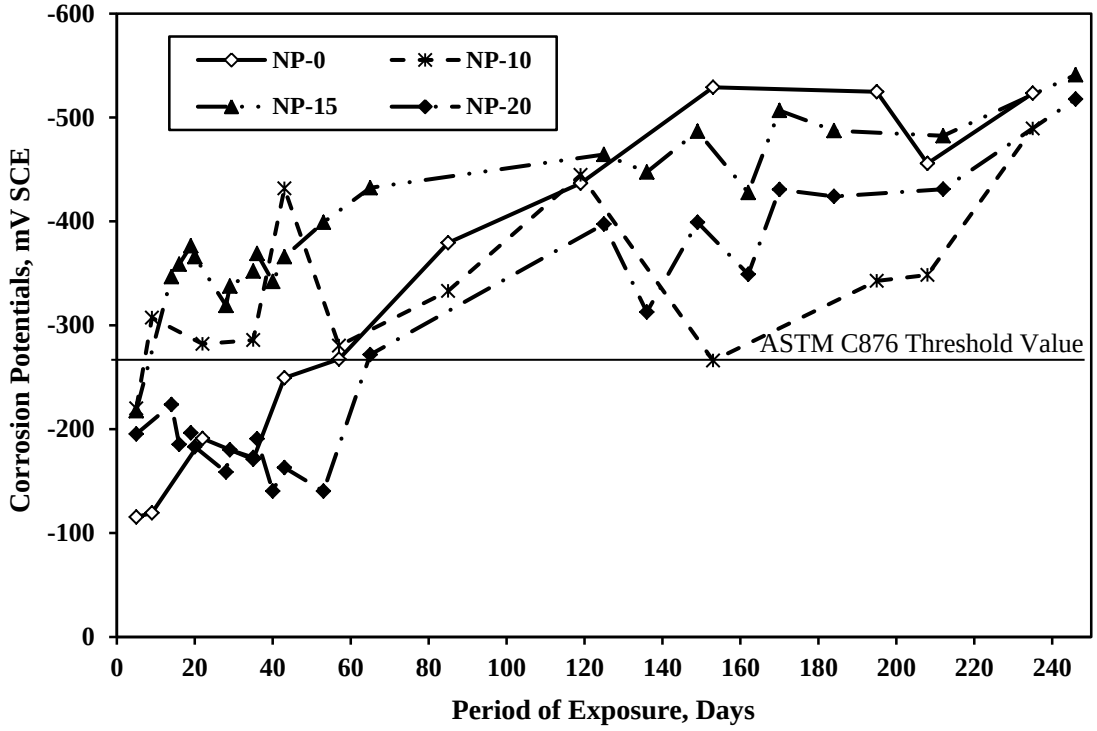

Figure 4-54: Corrosion Potentials on Steel in the Concrete Specimens Prepared with Different Percentages of NP and a w/cm Ratio of 0.55.

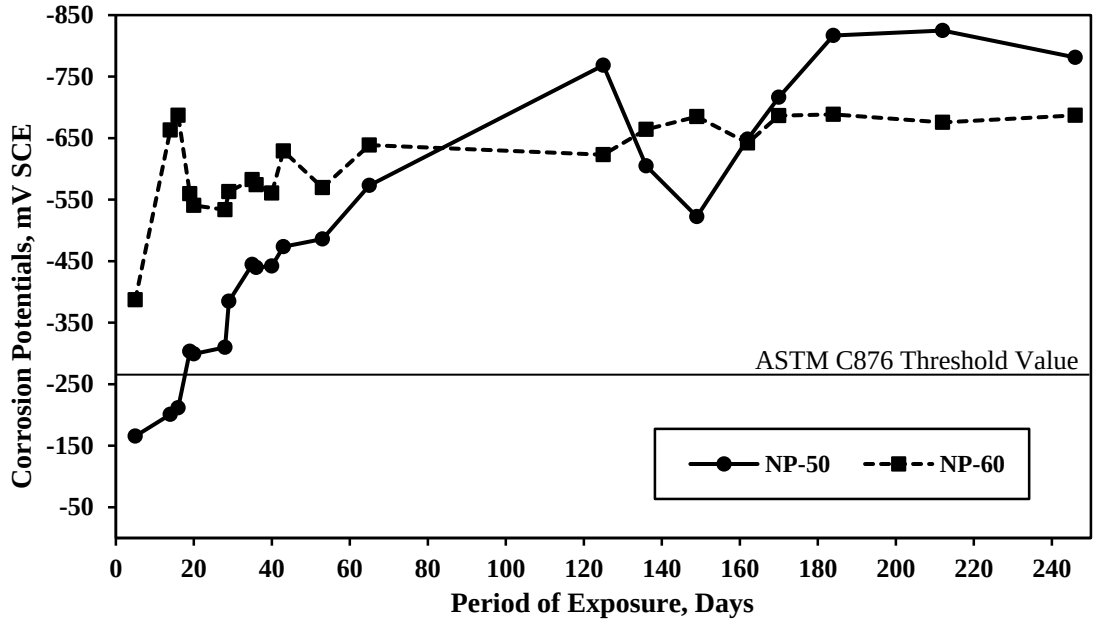

Figure 4-55: Corrosion Potentials on Steel in the Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

4.5.4 Corrosion Current Density

Concrete Specimens with a w/cm Ratio of 0.45

The corrosion current density (I_{corr}) on steel in the concrete specimens prepared with NP content varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.56. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. Lowest I_{corr} was noted in 10% NP cement concrete while it was almost the same in 15 and 20% NP cement concrete. The I_{corr} values in the NP cement concrete were lower than those in the plain cement concrete. After 300 days of exposure, the I_{corr} values on the 0, 10, 15 and 20% NP cement concretes are 0.80, 0.25, 0.26 and 0.24 $\mu A/cm^2$, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The I_{corr} in concrete specimens prepared using NP content 50 and 60%, as replacement of cement, is depicted in Figure 4.57. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. The I_{corr} of mix with 60% NP content was found to be more than 50% NP cement concrete. After 450 days of exposure, the I_{corr} values on the 50 and 60% NP cement concrete are 0.67 and 0.87 $\mu A/cm^2$, respectively.

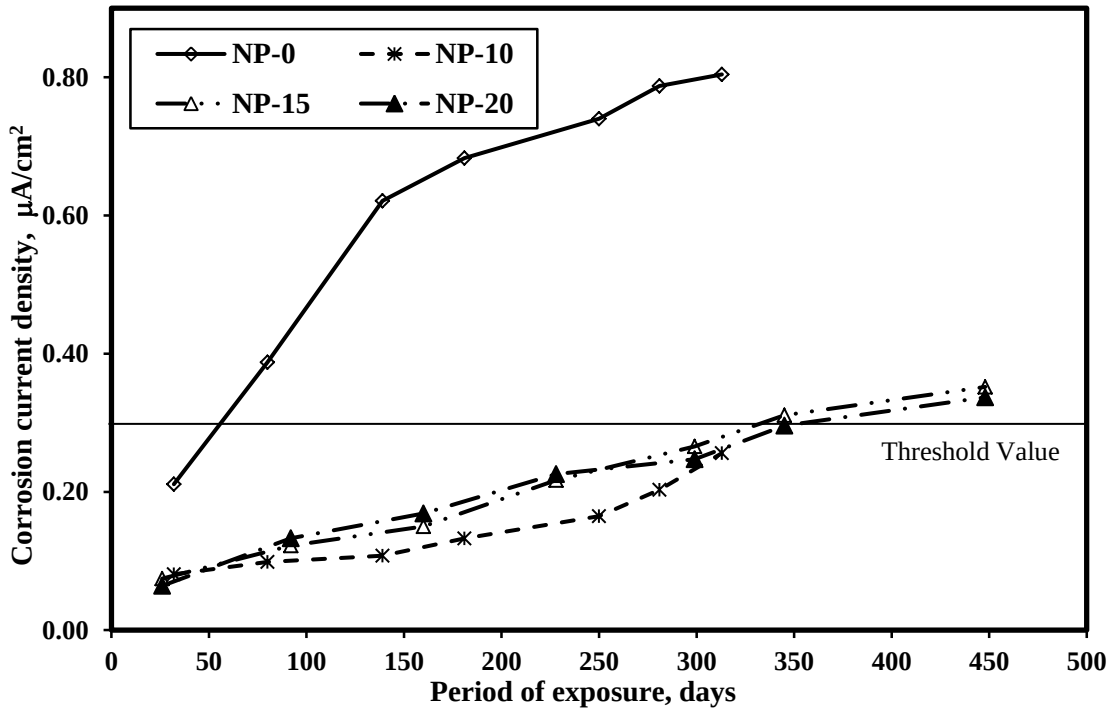

Figure 4-56: Corrosion Current Density on Steel in the Concrete Specimens Prepared with Different Percentages of NP and a w/cm Ratio of 0.45.

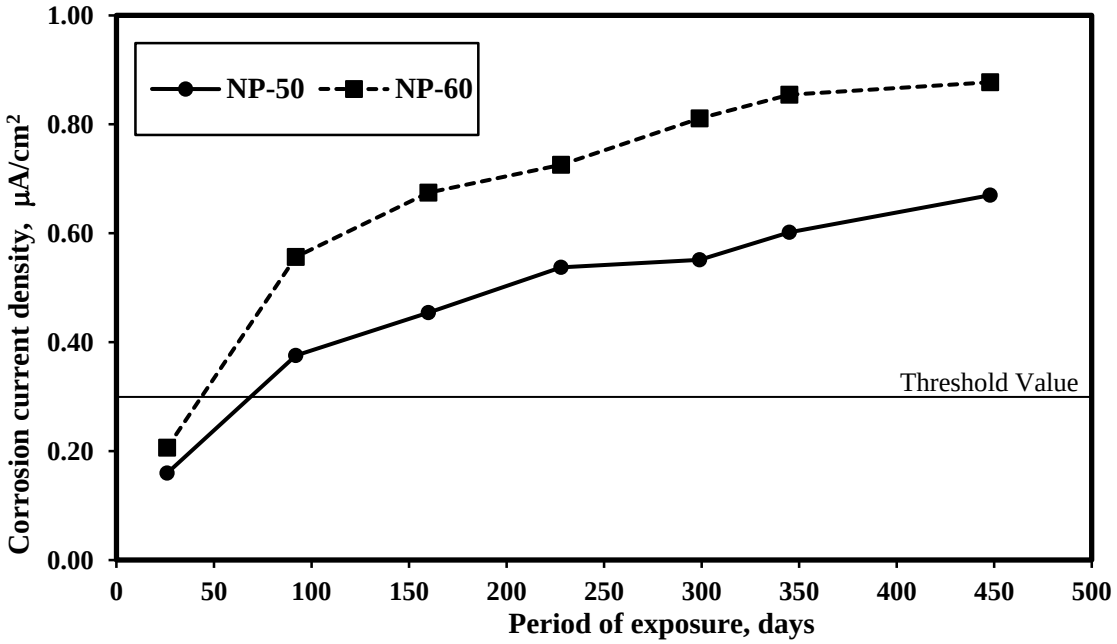

Figure 4-57: Corrosion Current Density on Steel in the Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

4.5.5 Electrical Resistivity

Concrete Specimens with a w/cm Ratio of 0.45

The electrical resistivity of concrete specimens prepared with NP content varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.58. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. The electrical resistivity was almost the same for 0, 10 and 20% NP cement concretes. Higher electrical resistivity was measured in the 15% NP cement concrete. The electrical resistivity at 3% moisture content for 0, 10, 15 and 20% NP cement concrete was 34.2, 36.9, 55.2, and 39.6 kΩ-cm, respectively. With regard to risk of reinforcement corrosion, summarised in Table 3.8, the specimens with 0, 10, and 20% NP content are in range of moderate intensity while the 15% NP cement concrete is in range of low intensity.

Concrete Specimens with a w/cm Ratio of 0.55

The electrical resistivity of concrete specimens prepared with 50 and 60% NP, as replacement of cement, is depicted in Figure 4.59. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. The electrical resistivity at 3% moisture content for 50 and 60% NP was 87.0 and 87.7 kΩ-cm, respectively. These values indicate that the risk of reinforcement corrosion of specimens with 50% and 60% NP are in the range of low intensity.

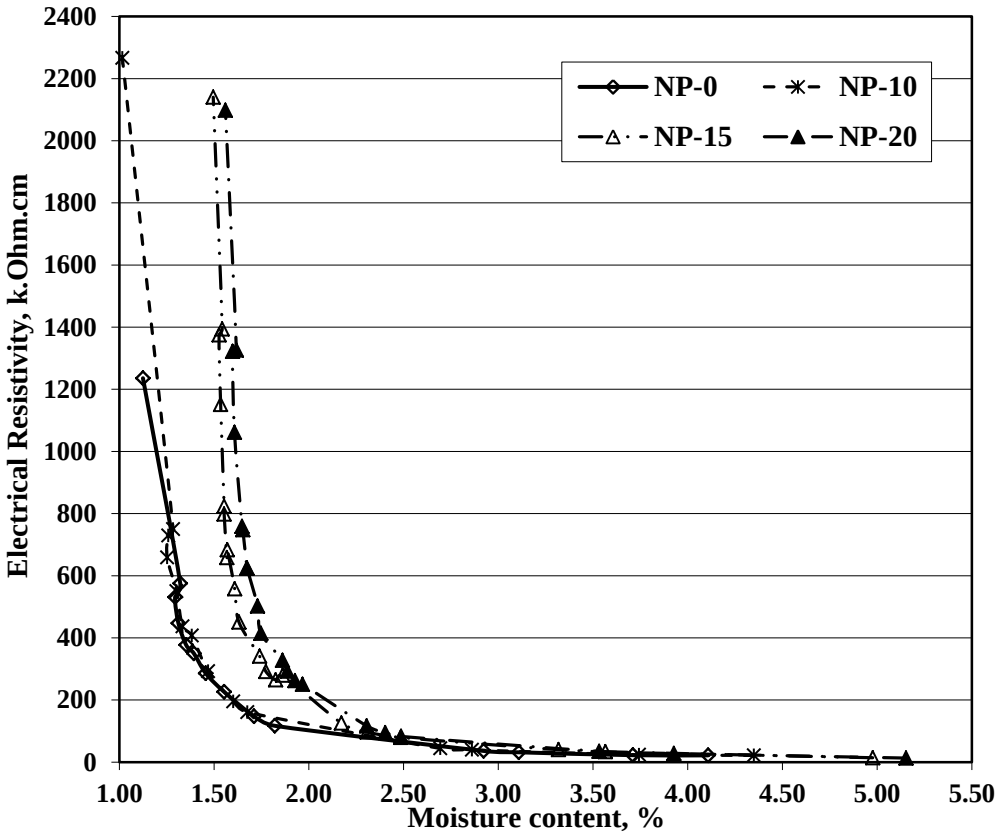

Figure 4-58: Electrical Resistivity of Concrete Specimens Prepared with Different Percentages of NP and a w/cm Ratio of 0.45 .

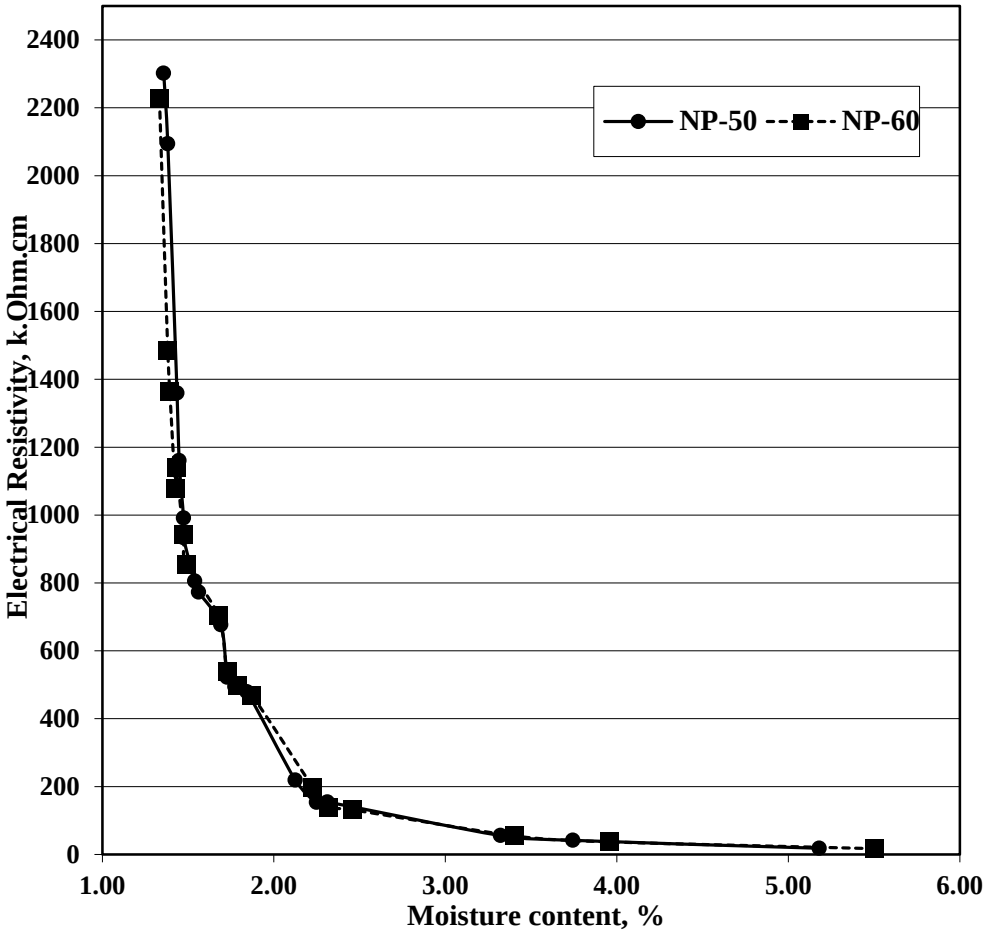

Figure 4-59: Electrical Resistivity of Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

4.5.6 Thermal Conductivity

Concrete Specimens with a w/cm Ratio of 0.45

The thermal conductivity of the concrete specimens prepared with NP varying from 0 to 20%, as replacement of cement, is depicted in Figure 4.60. Thermal conductivity decreased with an increase in the quantity of NP. However, the variation was not that significant. After

28 days of curing, the thermal conductivity of 0, 10, 15, and 20% NP was 2.4, 2.3, 2.3, and 2.3 W/mK, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The thermal conductivity of the concrete specimens with NP 50 and 60%, as replacement of cement, is depicted in Figure 4.61. The thermal conductivity of 60% NP cement concrete was more than that in the 50% NP concrete. After 28 days of curing, the thermal conductivity of 50 and 60% NP cement concrete was 2.2, and 2.4 W/mK, respectively.

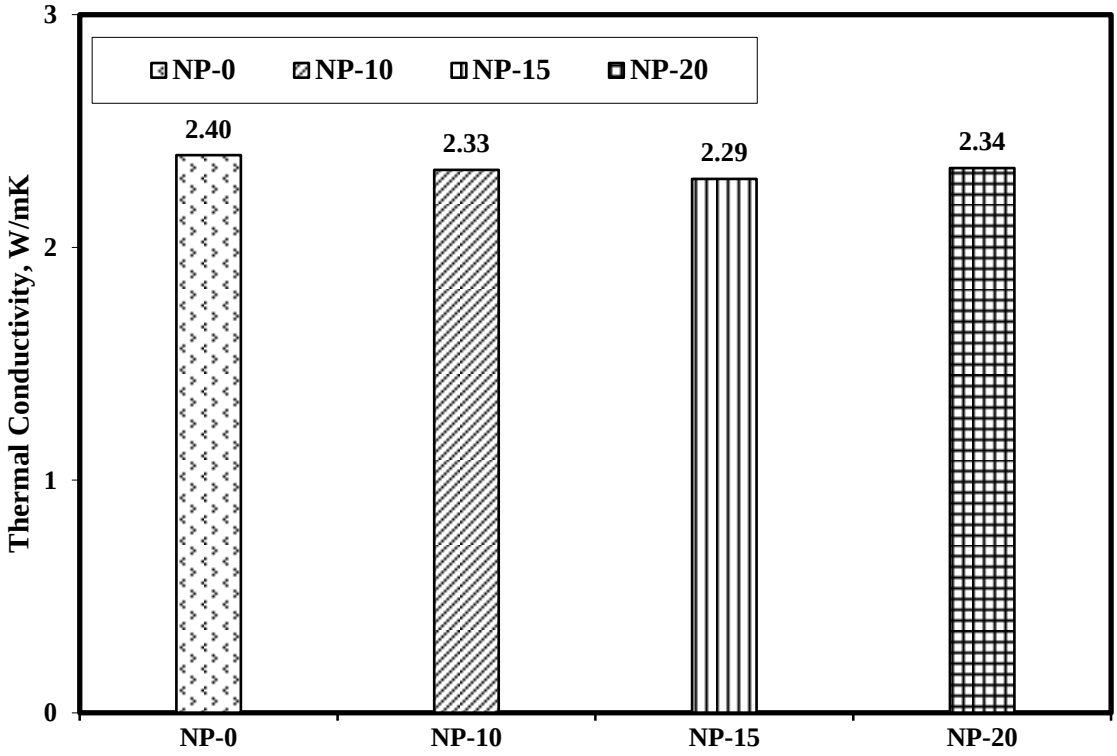

Figure 4-60: Thermal Conductivity of Concrete Specimens Prepared with Different Percentages of NP and a w/cm Ratio of 0.45.

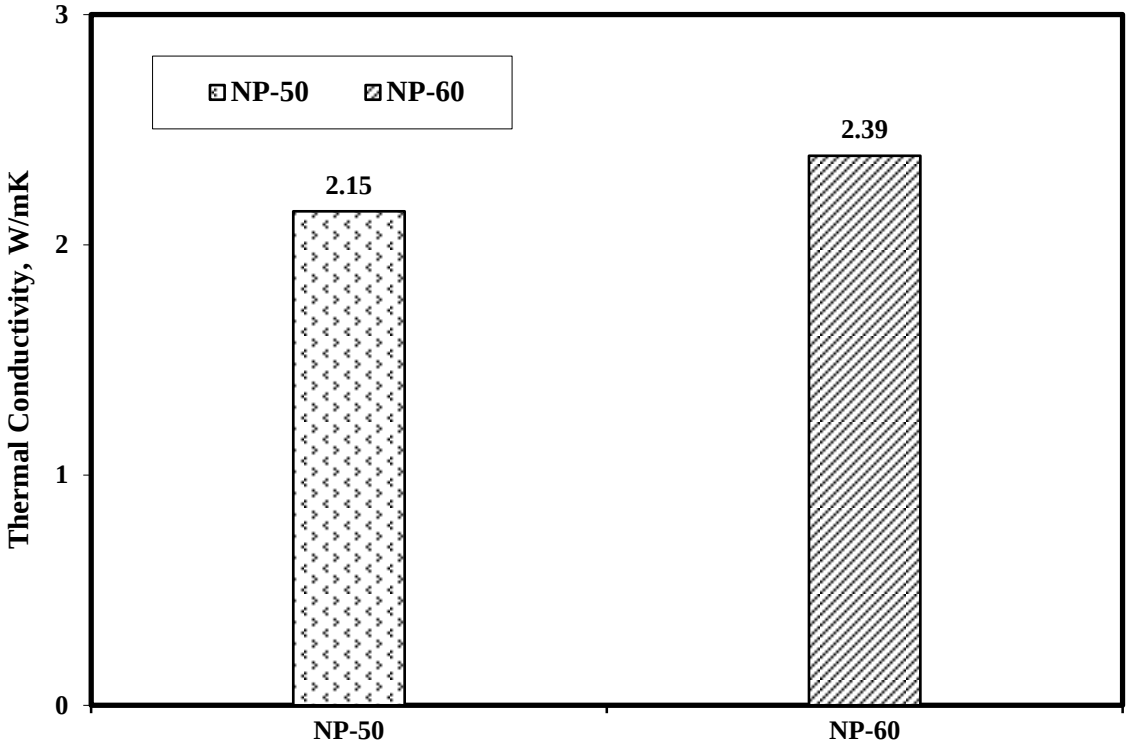

Figure 4-61: Thermal Conductivity of Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

4.5.7 Drying Shrinkage

Concrete Specimens with a w/cm Ratio of 0.45

The drying shrinkage in the concrete specimens prepared with NP content varying from 0 to 20%, is depicted in Figure 4.62. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, stabilizing with time and remaining almost unchanged thereafter. The drying shrinkage of all the NP cement concretes was less than that of plain cement concrete. Minimum shrinkage was noted in the 10% NP cement concrete. Drying shrinkage in the 15, and 20% NP was found to be almost similar. At 180 days, the

drying shrinkage of specimens with 0, 10, 15, and 20% NP cement concrete was 532, 322, 420, and 408 microns, respectively.

Concrete Specimens with a w/cm Ratio of 0.55

The drying shrinkage in the concrete specimens prepared with 50 and 60% NP, as replacement of cement, is depicted in Figure 4.63. The drying shrinkage increased with age in all the concrete specimens. The increase was more rapid initially, stabilizing with time. The drying shrinkage strain in the 50% NP cement concrete was less than that in the 60% NP cement concrete. After 180 days, the drying shrinkage of specimens 50 and 60% NP cement concretes was 485 and 631 microns, respectively.

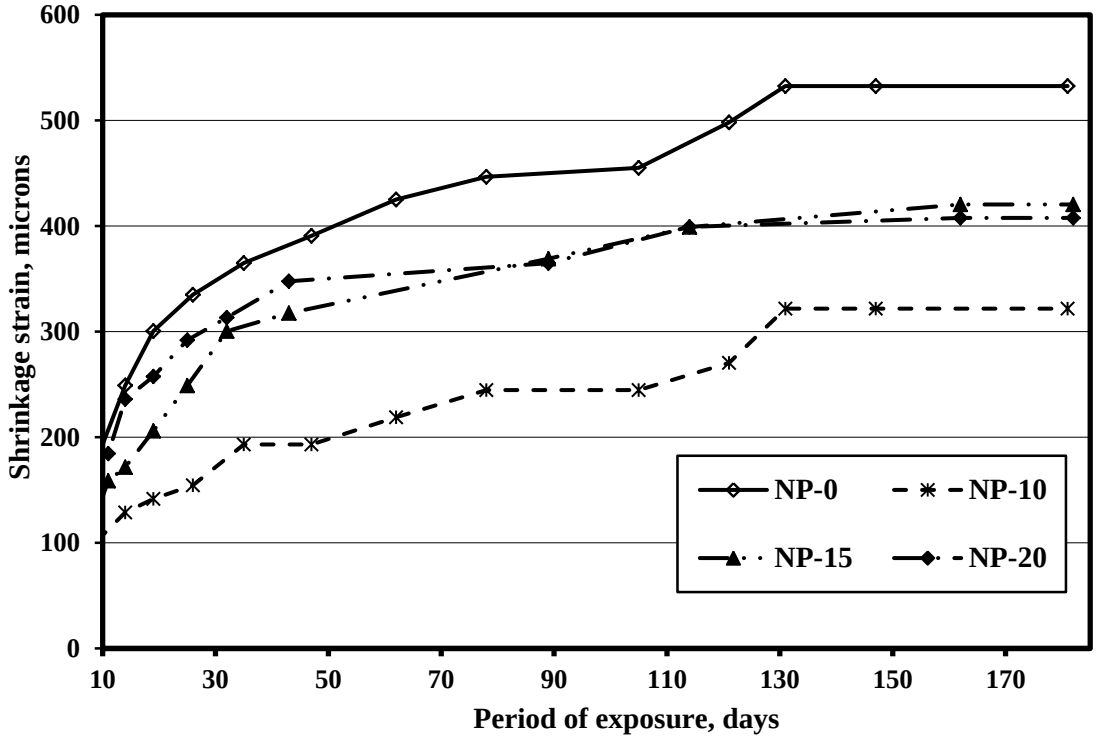

Figure 4-62: Drying Shrinkage Strain in Concrete Specimens Prepared with Different Percentages of NP and a W/C Ratio of 0.45.

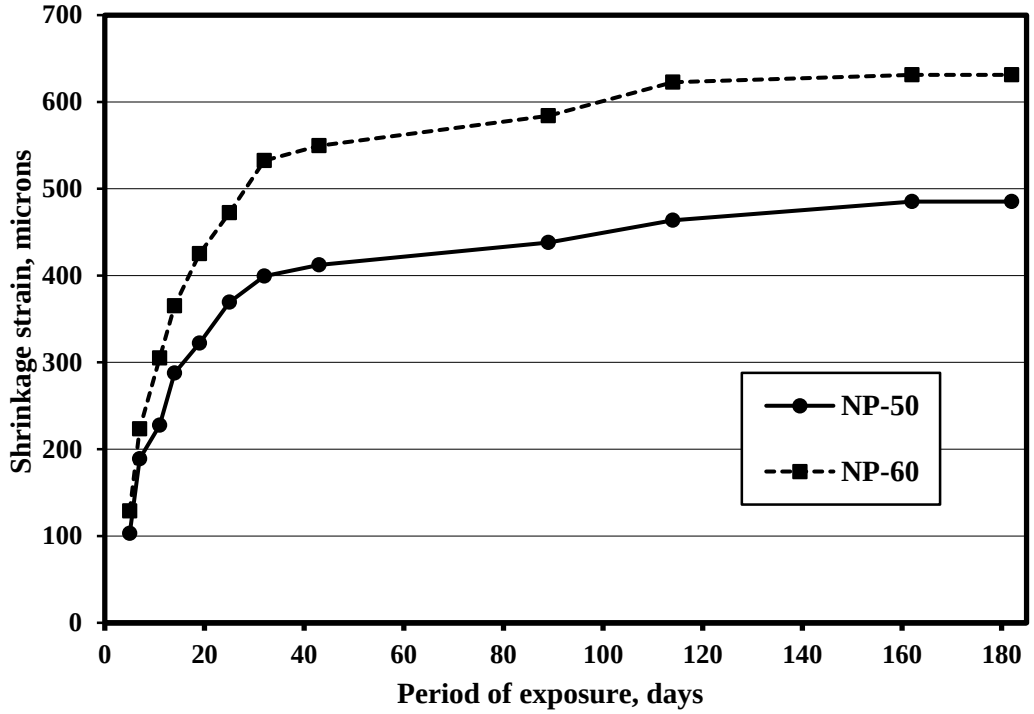

Figure 4-63: Drying Shrinkage Strain in Concrete Specimens Prepared with 50 and 60% NP and a w/cm Ratio of 0.55.

CHAPTER 5

DISCUSSION OF RESULTS

5.1 Properties of CKD Cement Concrete

5.1.1 w/cm ratio of 0.45

Table 5.1 summarizes the properties of CKD cement concrete specimens prepared with a w/cm ratio of 0.45 and 370 kg/m^3 cementitious materials content.

Table 5-1: Properties of CKD Cement Concrete Specimens with a w/cm ratio of 0.45.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} µA/cm^2	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
CONTROL	53.9	4.1	58	0.64	34.2	2.40	532
CKD-5	54.1	4.2	76	0.52	33.3	2.56	402
CKD-10	52.9	4.6	18	0.57	28.9	2.59	507
CKD-20	52.0	4.8	5	0.56	21.2	2.60	549

As stated earlier, four concrete mixtures were prepared with 0, 5, 10, 20% CKD and a w/cm ratio of 0.45 and cementitious materials content of 370 kg/m^3. The replacement of CKD had shown certain positive results. Maximum compressive strength was noted in 5% CKD cement concrete after 28 days of curing (i.e., 54.1 MPa). Further, the compressive strength decreased marginally with an increase in the CKD content in concrete as compared to 0 and 5% CKD cement concrete. However, it was observed that the compressive strength of 5, 10, and 20% CKD was not significantly varied from plain cement concrete (54.1, 52.9 and 52.0

for 5, 10 and 20% CKD). Hence, the early age strength of CKD cement concrete specimens was not much different from each other. The higher compressive strength of 5% CKD, compared to OPC concrete, may be attributed to an increase in the alkalinity of cement that accelerates the cement hydration. However, a further increase in the CKD content leads to reduction in strength, due to high replacement of cement and presence of higher content of alkali which modifies the nature and strength of the cement hydration products [85]. The effect of alkali in 5% CKD cement concrete was minimum while it increased with increase in the CKD content in concrete. Hence, 5% CKD can be considered as optimum from the point of view of compressive strength. The results in Table 5.1 indicate that 5 to 20% CKD could be utilized to produce high strength concrete since the strength was more than 50 MPa. Similar results were reported by Al-Harthy et al. [29] and Shoaib et al. [30] that the compressive strength development decreases with an increase in the CKD content.

The water absorption of 0 and 5% CKD cement concrete was almost similar. However, it increased marginally with further increase in the CKD. The minimum and maximum water absorption was observed in plain and 20% CKD cement concrete, respectively. However, the increase in water absorption when compared to plain cement concrete was not significant (4.2, 4.6 and 4.8% for 5, 10 and 20% CKD cement concrete). The increase in water absorption correlates with a reduction in the compressive strength.

The time to initiation of reinforcement corrosion in 5% CKD cement concrete was more than that in plain and other CKD cement concrete specimens. The time to initiation of reinforcement corrosion further decreased with an increase in the quantity of CKD (76, 18

and 5 days for 5, 10 and 20% CKD cement concrete). It was the minimum in the 20% CKD cement concrete.

Similarly, the minimum I_{corr} was measured on steel in the 5% CKD cement concrete (i.e., 0.52 $\mu A/cm^2$). While it was the maximum in the plain cement concrete specimens, the I_{corr} on steel in the other CKD cement concrete specimens was less than that of plain cement concrete. However, the difference in the I_{corr} values in 5, 10, and 20% cement concrete specimens was not that significant (0.57 and 0.56 $\mu A/cm^2$ for 10 and 20% CKD cement concrete). El-Sayed et al. [32] reported that up to 5% replacement of cement with CKD had no adverse effect on cement paste strength and on reinforcement passivity.

The data in Table 5.1 also indicate that the electrical resistivity was maximum in plain cement concrete. However, it decreased with an increase in the quantity of CKD. The electrical resistivity of 10 and 20% CKD was less than that of plain and 5% CKD cement concrete specimens. This may be attributed to the increase in the chloride concentration in the CKD concrete. However, for 3% moisture content, the risk of reinforcement corrosion was moderate for all CKD cement concrete specimens. Similar results have been reported by Maslehuddin et al. [33]. It should be noted that the I_{corr} in the CKD cement concrete was less than that of plain cement concrete. This indicates that the decrease in the electrical resistivity has a very small effect on the corrosion, particularly as the resistivity is in a similar range, i.e., moderate.

The thermal conductivity increased with an increase in the replacement of cement with CKD. The thermal conductivity in the plain cement concrete was less than that in CKD cement

concrete. Further, it increased with an increase in the quantity of CKD. However the increase in the thermal conductivity with an increase in the CKD content was not significant. The temperature gradient in concrete will be less due to low thermal conductivity, which minimizes the occurrence of micro-cracks in concrete structure. Thermal conductivity of mortar and cement paste are effected by the type of admixture and the amount of cement [86]. However, all the thermal conductivity values of CKD cement concrete specimens are in the typical range of 1.98–2.94 W/mK, specified by ACI for normal concrete [87].

Also, minimum drying shrinkage was observed in 5% CKD cement concrete (i.e., 402 microns). Further, the drying shrinkage increased with an increase in the quantity of CKD. The drying shrinkage of 5 and 10% CKD cement concrete specimens was observed to be less than that of 0% CKD cement concrete. The maximum drying shrinkage was recorded in 20% CKD concrete (i.e., 549 microns strain), possibly due to increase in the amount of fines in the specimen as the water required for workability will increase and leads to increased drying shrinkage. Also due to accelerated hydration of cement due to increase in the content of alkalinity. However, the variation in drying shrinkage of 0 and 20% CKD cement concrete was not significant. Maslehuddin et al. [8] reported that the shrinkage of CKD cement mortar increases with an increase in the quantity of CKD.

It is evident from the data in Table 5.1 that the performance of 5% CKD cement concrete was better than 0, 10 and 20% CKD cement concretes in terms of strength and most of the durability characteristics. Therefore, it would be rational to replace 5% cement with CKD. However, it should be noted that the properties of concrete produced with up to 20% CKD were not significantly different from that of 0% cement concrete. Therefore, the use of upto

20% CKD cement concrete should be encouraged. Since the 28-day compressive strength of 20% CKD concrete is more than 50 MPa, it can be used as structural concrete. This will lead to about 20% reduction in the consumption of cement.

5.1.2 w/cm of 0.55 (Low strength concrete)

Table 5.2 summarizes the properties of CKD cement concrete specimens prepared with a w/cm ratio of 0.55 and 300 kg/m^3 cementitious materials content.

Table 5-2: Properties of CKD Cement Concrete Specimens with a w/cm ratio of 0.55.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} $\mu A/cm^2$	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
CKD-50	29.5	5.6	5	0.72	17.7	2.46	545
CKD-60	21.0	6.2	5	0.66	15.2	2.09	627

The 50 and 60% CKD cement concrete mixtures were prepared with cementitious material content of 300 kg/m^3 and a w/cm ratio of 0.55. These were designed to produce low strength concrete with a low cementitious materials mixtures content and high w/cm ratio. However, it can be noted from the data in Table 5.2 that the compressive strength of 50 and 60% CKD cement concrete was in the low range (i.e., 20–30 MPa), the compressive strength of 50% CKD (29.5 MPa) was more than that of 60% CKD (21.0 MPa) cement concrete. This is expected due to the large reduction in the cement content and the increase in the w/cm ratio.

As the quantity of CKD was increased, the compressive strength, electrical resistivity and thermal conductivity decreased while the water absorption and I_{corr} and drying shrinkage

increased. However, the variation in these values was not that significant except for the compressive strength.

The data in Table 5.2 indicate that 50 and 60% CKD concrete with its 28-day compressive strength of about 29.5 and 21 MPa, can be used as low strength concrete. Being non-structural concrete, the typical applications are blinding concrete, filling trenches, pavements, etc. However, these concretes may need longer curing due to higher shrinkage (i.e., more than 500 microns), as noted in Table 5.2. Usage of 50% to 60% CKD in appropriate applications will decrease the cost and also lead to a reduction in consumption of cement.

5.2 Properties of LSP Cement Concrete

5.2.1 w/cm ratio of 0.45

Table 5.3 summarizes the properties of LSP cement concrete specimens prepared with a w/cm ratio of 0.45 and 370 kg/m^3 cementitious materials content.

Table 5-3: Properties of LSP Cement Concrete specimens with a w/cm ratio of 0.45.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} $\mu A/cm^2$	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
CONTROL	53.9	4.1	58	0.64	34.2	2.40	532
LSP-05	58.8	4.2	75	0.24	63.2	2.69	513
LSP-10	56.3	4.7	15	0.43	65.9	2.67	541
LSP-15	46.2	4.7	23	0.41	63.7	2.64	607

As stated earlier, four concrete mixtures were prepared with 0, 5, 10, 15% LSP and a w/cm ratio of 0.45 and cementitious material content of 370 kg/m^3. The compressive strength of

5% and 10% LSP cement concretes was more than that of plain cement concrete while the compressive strength of 15% LSP cement concrete was less than that of plain cement concrete. However, the variation in compressive strength of 5 and 10% LSP cement concretes was not that significant. Hence, 5% and 10% LSP cement concretes can be considered as high strength concretes since the strength was more than 50 MPa (i.e., 58.8 and 56.3 MPa for 5% and 10% LSP respectively). It was reported that LSP accelerates the hydration process at early ages (i.e., it does not play the role of inert material) [88]. Dhir et al. [41] reported that the compressive strength decreases with an increase in the quantity of LSP. However, they did not find much difference between 0 and 15% LSP cement concretes. Ramezanianpour et al. [89] did not note a significant reduction in the strength up to 10% replacement of LSP. However, they reported that the compressive strength decreased after 10% LSP [89]. They indicated that the reduction in compressive strength is the result of clinker dilution effect, which is a consequence of replacing a part of cement by the same quantity of LSP. The compressive strength of 15% LSP cement concrete can be considered as medium strength concrete since the strength was more than 30 MPa (i.e., 46.2 MPa).

The lowest water absorption was measured in 0% LSP i.e., 4.1%. Further, the water absorption increased with an increase in the LSP content. The maximum water absorption was observed in 10% and 15% LSP cement concrete (4.7% for both). Similar trend of water absorption was observed by Dhir et al. [41]. However, the variation in the water absorption in the 0 and 5% LSP cement concrete was marginal.

The electrical resistivity increased with an increase in the LSP content. This indicates good durability aspect of LSP cement concrete. The increase in electrical resistivity was significant

when compared to plain cement concrete (34.2, 63.2, 65.9 and 63.7 k.Ohm.cm for 0, 5, 10 and 15% LSP cement concrete). However, there was not much variation in values of 5, 10 and 15% LSP concrete specimens. Similarly, the I_{corr} in the LSP cement concrete specimens was less than that in the plain cement concrete specimens (0.64, 0.24, 0.43 and 0.41 $\mu A/cm^2$ for 0, 5, 10 and 15% LSP cement concrete). Minimum I_{corr} was noted in the 5% LSP cement concrete specimens. Hence, the 5% LSP displayed the best performance in term of corrosion resistance.

The thermal conductivity increased with an increase in the replacement level of cement with LSP. The maximum thermal conductivity was measured in 5% LSP cement concrete specimen. However, there was no significant variation in the 5, 10 and 15% LSP cement concretes. However, all the thermal conductivity values are in the typical range of 1.98 to 2.94 W/mK, specified by ACI for normal concrete [87].

The minimum drying shrinkage was noted in the 5% LSP cement concrete specimens (i.e., 513 microns). Further, it increased with an increase in the LSP content. The drying shrinkage of 10 and 15% LSP cement concretes was more than that of plain cement concrete (541 and 607 microns strain for 10% and 15% LSP cement concrete). However, there was no significant change in the drying shrinkage of 0, 5, 10% LSP cement concretes. From the data in Table 5.3, it is apparent that the 5% LSP performed better than plain cement concrete in terms of enhanced mechanical properties and durability characteristics. As such, the use of 5% LSP is recommended for high strength and durability requirements. Since 10% LSP has also exhibited high strength (more than 55 MPa), it can also be considered for high strength applications, but at locations not requiring high durability. Concrete with 15% LSP can be

used as medium strength concrete with medium durability requirements. Such classified use of LSP, which is abundantly available in the Kingdom, will lead to a significant savings in cost of concrete and decreases the consumption of cement.

5.2.2 w/cm ratio of 0.55

Table 5.4 summarizes the properties of LSP cement concrete specimens prepared with a w/cm ratio of 0.55 and 300 kg/m^3 cementitious material content.

Table 5-4: Properties of LSP Cement Concrete Specimens with a w/cm ratio of 0.55.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} μA/cm^2	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
LSP-50	18.6	6.0	z	0.58	102.4	2.96	755
LSP-60	13.9	6.3	5	0.51	76.2	2.77	776

The cement concrete mixtures with 50 and 60% LSP were prepared with cementitious material content of 300 kg/m^3 and a w/cm ratio of 0.55. A decrease in strength and change in other properties can be observed by increasing the LSP from 50 to 60%. The compressive strength decreased from 18.6 to 13.9 MPa with an increase in the LSP content. These mixes were designed to produce low strength concrete with low cementitious materials content and high w/cm ratio. Hence, the compressive strength was expected to be in the low range.

The water absorption also increased with an increase in the LSP content. Similar results have been reported by Dhir et al. [41]. The maximum water absorption of 6.3% was measured in 60% LSP cement concrete specimens. However, the increase was not that significant.

The electrical resistivity of 60% LSP cement concrete was less than that of 50% LSP cement concrete (102.4 and 76.2 k.Ohm.cm for 50 and 60% LSP cement concrete). However, for a moisture content of 3%, the risk of corrosion was low for both the concrete specimens. The electrical resistivity for 50 and 60% LSP cement concrete was more than the values mentioned in the Table 5.3.

The I_{corr} on steel in the concrete specimen with 50% LSP (i.e., 0.58 $\mu A/cm^2$) was slightly more than that on steel in 60% LSP cement concrete (i.e., 0.51 $\mu A/cm^2$). However, corrosion initiation was noted in the both specimens, since they surpassed the corrosion threshold of 0.3 $\mu A/cm^2$, as depicted in Figure 4.22.

The thermal conductivity of concrete specimen with 50% LSP (2.96 W/mK) was slightly more than that of specimens with 60% LSP (2.77 W/mK). The 50% LSP cement concrete was more efficient in transmitting heat.

The drying shrinkage in both 50 and 60% LSP cenment concrete (i.e., 755 and 776 microns, respectively) was almost similar and more than 500 μm that is generally considered to be a threshold value.

The low compressive strength of 50 to 60% LSP cement concrete makes the usage of LSP material in the typical low strength applications that include: blinding concrete, trench fill. Low cementitious materials content and 50-60% replacement of cement with LSP considerably reduces the cost of this material.

5.3 Properties of EAFD Cement Concrete

5.3.1 w/cm ratio of 0.45

Table 5.5 summarizes the properties of EAFD cement concrete specimens prepared with a w/cm ratio of 0.45 and 370 kg/m^3 cementitious materials content.

Table 5-5: Properties of EAFD Cement Concrete Specimens with a w/cm ratio of 0.45.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} μA/cm^2	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
CONTROL	53.9	4.1	58	0.64	34.2	2.40	532
EAFD-05	54.7	4.3	19	0.52	38.8	2.43	513
EAFD-10	42.4	4.6	23	0.61	39.2	2.45	545
EAFD-15	47.9	4.9	9	0.64	39.2	2.52	600

The maximum compressive strength was observed in 5% EAFD cement concrete. Further, the strength decreased with an increase in the EAFD content. The compressive strength of 5% EAFD cement concrete (i.e., 54.7 MPa) was slightly more than that of plain cement concrete (53.9 MPa). From Figure 4.29, it can be noted that the 90 day compressive strength of 5% EAFD was slightly more than that of plain cement concrete. The compressive strength reduced with a further increase in the EAFD content. A similar trend was reported by Alexandre et al. [36] and Guray et al. [84]. According to Alexandre et al., the reduction in compressive strength may be attributed to a delay in the hydration reactions caused by EAFD in the initial stages, as noted from the heat of hydration measures. With this theory, it was observed that the compressive strength of 5% EAFD cement concrete gains significant strength between 3 and 7 days. There was no significant gain in early compressive strength of 10 and 15% EAFD. However, it may increase after longer curing durations.

Due to presence of Zn and Pb in EAFD, the formation of a membrane around cement particles with the precipitation of calcium hydroxyzincate ($CaZn_2$ $(OH)_6$ H_2O) can prevent water and ion transport needed for cement hydration [84]. Consequently, the retardation or suppression of cement hydration adversely affects the strength development [84]. The 90 day strength of 10% EAFD cement concrete (i.e., 52.0 MPa) was slightly more than that of 15% EAFD cement concrete (i.e., 50.5 MPa).

The water absorption after 28 days of curing increased with an increase in the quantity of EAFD. However, the difference in the water absorption of 0 and 5% EAFD cement concrete was marginal. The maximum water absorption was noted in 15% EAFD cement concrete specimens (i.e., 4.9%). As discussed above, at 28 days, this high absorption might be the effect of delay in hydration reactions caused by EAFD at initial stages.

The electrical resistivity increased with an increase in the quantity of EAFD. The electrical resistivity of 10 and 15% EAFD cement concrete (i.e., 39.2 k.Ohm.cm) was same and more than that of other cement concretes; including plain cement concrete. For the 3% moisture content, the risk of reinforcement corrosion was of moderate range.

The I_{corr} on steel in EAFD cement concrete specimens was less than that of plain cement concrete. The minimum I_{corr} was measured in 5% EAFD cement concrete (i.e., 0.52 $\mu A/cm^2$). Further, it increased with an increase in the EAFD content in concrete. The I_{corr} of 15% EAFD was almost the same as that in the plain cement concrete (i.e., 0.64 $\mu A/cm^2$). Hence, 5% EAFD cement concrete was observed advantageous, compared to all EAFD cement concrete specimens. Al-Mutlaq and Chaudhary [90] reported that the EAFD

enhances the ability to bind chlorides and increases its alkalinity due to the increase in the hydroxyl ion concentration in pore solution and this will increase with hydration time. This process will result in reduction of Cl^-/OH^- ratio, which indicates that the risk of chloride-induced corrosion of reinforcing steel is reduced with EAFD addition in concrete.

The thermal conductivity increased with an increase in the quantity of EAFD (2.40, 2.43, 2.45 and 2.52 W/mK for 0, 5, 10 and 15% EAFD cement concrete). However, the increase was not that significant. The thermal conductivity of concrete is generally influenced by its density, aggregate type, and the moisture content [91]. The delay in the formation of hydration products might have kept the concrete moist from inside therby increasing the thermal conductivity. According to Uysal et al. [92] the thermal conductivity of concrete increases with an increase in the moisture content of concrete, However, all the thermal conductivity values of specimens were in the typical range of 1.98–2.94 W/mK, specified by ACI [87].

Minimum drying shrinkage was measured in the 5% EAFD cement concrete specimens (i.e., 513 microns). Further, the drying shrinkage increased with an increase in the EAFD content in concrete. The drying shrinkage of 5% EAFD was less than that of 0% EAFD cement concrete. Whereas, for 10 and 15% EAFD cement concrete, it was more than plain cement concrete (545, 600 and 532 microns for 10, 15 and 0% EAFD cement concrete). The increase in drying shrinkage may be ascribed to the presence of more fines in specimens.

The data in the Table 5.5 indicate that up to 5% EAFD cement concrete can be used as high strength concrete. While the 10 and 15% EAFD cement concretes can be used as medium

strength concrete, the use of up to 15% EAFD will result in a cost saving of concrete and reduction in the cement consumption. The durability characteristics of 5% EAFD cement concrete are similar to those of OPC concrete while those of 10 and 15% EAFD cement concretes are slightly lower than those of OPC.

Initial and Final Setting Time of Cement Blended With Normal EAFD Cement Concrete:

Referring to Table 4.1, it was observed that the initial and final setting time of normal EAFD cement concrete increased significantly with an increase in EAFD content in the specimens. The limit for initial and final setting time specified by the ASTM C 150 is 45 and 375 minutes, respectively. However, all EAFD cement specimens exceeded the limit in the final setting time. The final setting was 265, 415, 615, 1123, 1470, 1672 minutes for 0, 5, 10, 15, 50 and 60% EAFD cement, respectively.

According to de-Souza et al. [34], the delay in setting time of samples with EAFD could be due to the presence of zinc. The zinc in EAFD changes to amorphous hydroxide, $Zn(OH)_2$, which forms a protective coating on the surface of anhydrous cement grains. This coating formation delays the hydration of cement, mainly the C_3S phase. After this delay, high Ca^{2+} and OH^- concentrations enable the zinc hydroxide to transform into zinc calcium hydroxide, which is a crystalline compound. Thereafter, the hydration process starts again.

Initial and final setting time of Cement Blended with Thermally Treated EAFD

As discussed earlier, the setting time increased considerably with an increase in the EAFD content in cement. Hence, to reduce the setting time, the EAFD was thermally treated and the optimistic results could be depicted in Table 4.2 for 50 and 60% EAFD. It was witnessed from Table 4.2 that the setting time of 50 and 60% thermally treated EAFD decreased significantly when compared to normal EAFD. The final setting time for 50% EAFD cement decreased from 1470 to 415 minutes while for 60% EAFD cement, it decreased from 1672 to 615 minutes. The EAFD was heated for about 2 hours at a temperature of 900^0C. The thermal treatment leads to change in the properties of EAFD. Lee et al. [93] reported that the volatilization of lead, zinc and cadmium increased in proportion to the temperature. He observed that the volatilization of cadmium finished at 800^0 C, and those of lead and zinc were finished at 1000^0 C. This results in the reduction of zinc content in EAFD. Therefore, there will be not enough zinc content for precipitation of calcium hydroxyzincate to form a membrane around cement particles. Zinc was the main cause for the delay in the setting time, since it got volatilized and the setting time of EAFD cement concrete specimens decreased.

5.3.2 w/cm ratio of 0.55

Table 5.6 summarizes the properties of EAFD cement concrete specimens prepared with a w/cm ratio of 0.55 and 300 kg/m^3 cementitious materials content.

Table 5-6: Properties of EAFD Cement Concrete Specimens with a w/cm ratio of 0.55.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} $\mu A/cm^2$	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
EAFD-50	17.1	5.5	5	0.66	32.0	2.57	624
EAFD-60	14.2	5.5	5	0.63	26.8	2.34	786

From the data in Table 5.6, it is evident that the compressive strength significantly decreased with an increase in the EAFD content in concrete (17.1 MPa). The maximum compressive strength was recorded in 50% EAFD cement concrete. The magnitude of compressive strength reduction when compared to earlier EAFD mixes (5, 10 and 15%) was due to a reduction in the quantity of cement and an increase in w/cm ratio.

Reduction in compressive strength with an increase in the quantity of EAFD followed the trend of earlier research work [36, 84]. The water absorption was same in 50 and 60% EAFD cement concrete with an increase in the EAFD content (i.e., 5.5%).

The corrosion current density decreased with increase in EAFD content; however, the decrease was not that significant (0.66 and 0.63 $\mu A/cm^2$ for 50 and 60% EAFD cement concrete). However the 50% EAFD has shown better performance in terms of electrical resistivity when compared to the 60% EAFD concrete (32.0 and 26.8 k.Ohm.cm for 50 and 60% EAFD).

Thermal conductivity decreased with an increase in the EAFD content (2.57 and 2.34 for 50 and 60% EAFD). However, the decrease was not that significant. The minimum thermal

conductivity was recorded in 60% EAFD cement concrete. However, the thermal conductivity values were in the typical range of 1.98–2.94 W/mK, specified by ACI [87].

Drying shrinkage in 60% EAFD was more than that of 50% EAFD (624 and 786 microns for 50 and 60% EAFD). Drying shrinkage increased with an increase in the EAFD content. Also, it should be noted that the drying shrinkage in both 50% and 60% EAFD cement concretes was more than the acceptable value of 500 μm [14]. Therefore, appropriate curing is necessary to avoid drying shrinkage cracking in these concretes.

Since the compressive strength of 50 and 60% EAFD cement concretes is in the range of 17 to 14 MPa, it can be used as a very low strength concrete. Typical applications include: blinding concrete and non-structural applications, such as pavements, blocks, etc.

Leachability of Heavy Metals

Leachability of heavy and hazardous metals is used to assess the potential for leaching of metals into ground and surface waters, and provides an assessment of metal mobility under actual field conditions, such as its effect upon rain. The EAFD should not leach any toxic and harmful substances into the surrounding environment, when replaced with cement in concrete specimens. TCLP method [81] was used to determine the release of harmful substances from EAFD cement concrete specimens into the service environment.

The TCLP set by the United States Environmental Protection Agency (USEPA) was performed on all EAFD specimens. The eight hazardous USEPA regulated TCLP metals are arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver. The specimens for

leaching analysis were prepared and made ready for the TCLP tests that were conducted at the Center of Environment and Water, Research Institute.

The results of leaching test carried out on EAFD cement concrete specimens after 28 days of curing are presented in Table 4.4. It can be depicted from the data in Table 4.4 that the concentration of zinc, barium and chromium were found to be less than the allowable limits for toxicity characteristics. Therefore, the results shown in Table 4.4 indicate that all EAFD specimens can be classified as non-hazardous. Hence, it is concluded that EAFD cement concrete specimens will not be any threat to the environment.

5.4 Properties of OA Cement Concrete

5.4.1 w/cm ratio of 0.45 (Medium strength concrete)

Table 5.7 summarizes the properties of OA cement concrete specimens prepared with a w/cm ratio of 0.45 and 370 kg/m^3 cementitious materials content.

Table 5-7: Properties of OA Cement Concrete specimens with a w/cm ratio of 0.45.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} μA/cm^2	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
CONTROL	53.9	4.1	58	0.64	34.2	2.40	532
OA-05	55.5	4.4	8	0.48	103.4	2.51	489
OA-10	43.9	5.2	5	0.55	120.1	2.37	502

As stated earlier, three concrete mixtures were prepared with 0, 5 and 10% OA and a w/cm ratio of 0.45 and cementitious materials content of 370 kg/m^3. From the data in Table 5.6, it is evident that the 5% OA cement concrete has the maximum compressive strength (i.e., 56

MPa). However, it decreased with an increase in the OA content in the concrete. Further, the compressive strength of 5% OA cement concrete was slightly more than that of the control mix, whereas the compressive strength of 10% OA was less than that of 0 and 5% OA cement concretes (53.9 and 43.9 MPa for 0 and 10% OA). It was reported that the compressive strength of 5 and 7% OA was more than that of control specimen while that of the 10% OA was less than that 0% OA cement concrete [16]. Paya et al. [94] reported that the compressive strength development of OA cement concrete decreased significantly with an increase in the OA content in the mixture. According to Paya et al. [94] the magnesium compounds present in the OA could produce damage in the cement matrix due to cement reactions that lead to inorganic corrosion reactions of cement matrix which results in loss of bonding capacity of material and a decrease in mechanical and durability properties [94]. The compressive strength of 5% OA was almost the same as that plain cement concrete after 14 days of curing. Whereas, the early age compressive strength of 10% OA cement concrete was much less than that of the plain cement concrete. Hence, there was improvement in compressive strength development after 28 days of curing.

The water absorption of OA cement concrete specimens increased with an increase in the quantity of OA (4.1, 4.4 and 5.2% for 0, 5 and 10% OA cement concrete). The water absorption of 5% OA was slightly more than that of the control mix. Maximum water absorption was measured in 10% OA cement concrete. This is in agreement with the compressive strength of OA specimens after 28 days of curing.

The I_{corr} in the 5 and 10% OA cement concretes was less than that of the control concrete (0.64, 0.48 and 0.55 $\mu A/cm^2$ for 0, 5 and 10% OA). Minimum I_{corr} was noted in the 5% OA cement concrete. Further, it increased with an increase in the OA content in concrete.

The data on electrical resistivity shows a trend similar to that of corrosion current density. The electrical resistivity of plain cement concrete is almost one-third of the OA cement concrete (34.2, 103.4 and 120.1 k.Ohm.cm for 0, 5 and 10% OA). The electrical resistivity of control specimen is in the medium range while it is in the low range in the OA cement concretes.

The thermal conductivity of control and OA cement concrete is almost the same i.e., 2.5 W/mK, being in the range of 2.37 to 2.51 W/mK with minimal variation between the three concrete types.

The drying shrinkage of OA cement concrete specimens decreased with an increase in the OA content (532, 489, and 502 microns strain for 0, 5 and 10% OA). However, the drying shrinkage of 10% OA was slightly more than that of 5% OA cement concrete. Paya et al. [94] reported that the expansive reactions in cement paste will occur due to hydration of magnesium oxide or due to reaction of sulfates with calcium aluminate and this will not undergo any appreciable volume change. They found that no expansive processes were detected in OA cement pastes by the Le Chatelier needles test. The cracks due to expansion process will not occur when using OA cement concrete and hence the structural stability due to internal attack is guaranteed.

Leachability of Heavy Metals

The OA contains heavy metals (e.g. vanadium and nickel) which are contained in the crude petroleum at the outset as shown in Table 4.5 and they are increased during the process of incineration. The high carbon content and presence of toxic heavy metals suggested that this OA can be considered as an hazardous respirable. Hence, TCLP method was used to determine the discharge of harmful substances from OA cement concrete specimens into the surroundings.

The specimens for leaching analysis were prepared and made ready for the TCLP tests that were conducted at the Center of Environment and Water, Research Institute.

The results of leaching test carried out on OA cement concrete specimens after 28 days of curing are presented in Table 4.5. It can be depicted from the data in Table 4.5 that the concentration of vanadium, barium and chromium were below the allowable limits for toxicity characteristics. Therefore, the results shown in Table 4.5 indicate that all OA cement concrete specimens can be classified as non-hazardous. Hence, it is concluded that OA cement concrete specimens are not detrimental to the environment.

The data in Table 5.7 indicate that upto 5% OA cement concrete can be used as high strength. Such a concrete will have a 28-day compressive strength of more than 55 MPa and low corrosion current density. The drying shrinkage of this concrete will be less than 489 microns. The use of such a concrete for structural purpose will result in cost saving and a reduction in the consumption of cement.

5.5 Properties of NP Cement Concrete

5.5.1 w/cm ratio of 0.45

Table 5.8 summarizes the properties of NP cement concrete specimens prepared with a w/cm ratio of 0.45 and 370 kg/m^3 cementitious materials content.

Table 5-8: Properties of NP Cement Concrete Specimens with a w/cm ratio of 0.45.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} μA/cm^2	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
CONTROL	53.9	4.1	58	0.64	34.2	2.40	532
NP-10	51.9	4.4	8	0.11	36.9	2.33	322
NP-15	46.9	5.0	10	0.15	55.2	2.29	415
NP-20	44.6	5.2	65	0.16	39.6	2.34	406

NP is very prominent for its beneficial utilization in concrete for its advantageous properties such as low heat of hydration, high ultimate strength, high sulfate resistance, low permeability, and low alkali–silica activity [18]. From the data in the Table 5.8, it may be noted that the compressive strength of 0% NP was the maximum (53.9, 51.9, 46.9 and 44.6 MPa for 0, 10, 15 and 20% NP). As expected, the compressive strength decreased with an increase in the NP content in concrete. Minimum compressive strength was noted in 20% NP cement concrete, the reduction was almost 17% when compared to 0% NP cement concrete. However, after 90 days, the compressive strength of 10% NP cement concrete was more than that of the control specimens. This may be attributed to the pozzolanic reaction that continues with age. However, the compressive strength of 15 and 20% NP cement concretes was less than that of control specimen after 90 days. This result is in agreement

with several researchers who reported that the compressive strength decreases with an increase in the replacement level of cement with NP [18, 46-48, 95-98]. Replacing cement with NP, especially in high volume substitution, caused a significant decrease in the heat produced during hydration [96]. Fajardo et al. [48] reported that the early reduction in strength is due to the slow chemical reactions of NP. Favorable temperatures and continuous curing for a long period of time may be needed for the completion of pozzolanic reaction in NP.

Since the early chemical reactions are slower and densification of NP cement concrete can be observed after advanced age, the water absorption of NP cement concrete specimens after 28 days of curing increased with an increase in the replacement level of cement with NP (4.1, 4.4, 5.0 and 5.2 for 0, 10, 15 and 20% NP cement conrete). However, the increase was marginal for the 10% NP cement concrete.

The most important durability problem of reinforced concrete is corrosion of steel in concrete. The durability of reinforced concrete structures is guaranteed by both the physical and chemical protection of the steel against corrosion. The delay in initiation of reinforcement corrosion was observed in 20% NP cement concrete (i.e., 65 days). A similar trend was also reported by Fajardo et al. [48] However, the I_{corr} in all the NP cement concretes was much less than that in the plain cement concrete (0.64, 0.11, 0.15, and 0.16 for 0, 10, 15 and 20% NP). The I_{corr} of the NP cement concretes was less than that of plain cement concrete. However, it increased marginally with an increase in the quantity of NP. It may be noted from Figure 4.56 that the I_{corr} was less than the threshold value even after 300

days. Hence, the NP cement concrete specimens have shown the beneficial effect on rate of reinforcement corrosion. Similar results were reported in an earlier research [16].

It is known that the electric resistivity of concrete is influenced by the porosity, moisture content and the ionic concentration of the pore solution. The resistivity of concrete as a function of moisture content was presented in Figure 4.58. The electrical resistivity of concrete specimens increased with an increase in the quantity of NP. However, the electrical resistivity of 15% NP cement concrete was less than that of 10% NP cement concrete (i.e., 2.33 and 2.29 for 10 and 15% NP). Fajardo et al. [48] reported that the electrical resistivity increased with an increase in the quantity of NP. The use of pozzolanic materials reduces the porosity of the NP cement concrete which is directly related to electrical resistivity [48]. The concrete specimens with 0, 10 and 20% NP are in range of moderate risk of reinforcement corrosion, while 15% NP cement concrete specimens was under low risk of reinforcement corrosion.

The thermal conductivity of concrete increased with an increase in the cement dosage and density [92]. From the data in Table 5.7, it can be observed that the thermal conductivity of NP cement concrete decreased with an increase in the quantity of NP in the concrete (2.40, 2.33, 2.29 and 2.34 W/mK for 0, 10, 15 and 20% NP). However, there was no significant difference in the thermal conductivity of plain and NP cement concretes, whereby the values being in the range of 2.29 to 2.40 W/mK.

The drying shrinkage of NP cement concrete specimens decreased with an increase in the quantity of NP, as shown in Table 5.8. The drying shrinkage of 15 and 20% NP was almost

same (i.e., 415 and 406 microns for 15 and 20% NP), while the minimum shrinkage was noted in the 10% NP cement concrete (i.e., 322 microns strain). These results are in line with data reported in an earlier study [16].

From the data in Table 5.8 it is evident that up to 10% NP can be used to produce high strength with a compressive strength of more than 50 MPa. Such a concrete will provide extended service life since the corrosion current density is low and drying shrinkage is lower than the other mixes. The compressive strength of 15 and 20% NP cement concrete was about 45 MPa. Consequently, they can be used as medium strength concrete.

5.5.2 w/cm ratio of 0.55

Table 5.9 summarizes the properties of NP cement concrete specimens prepared with a w/cm ratio of 0.55 and

Table 5-9: Properties of NP Cement Concrete Specimens with a w/cm ratio of 0.55.

Mix designation	28 days compressive strength, MPa	28 days water absorption, %	Time to initiation of reinforcement corrosion, days	I_{corr} $\mu A/cm^2$	Electrical resistivity at 3% moisture k.Ohm.cm	Thermal conductivity, W/mK	5 months Drying shrinkage strain, Microns
NP-50	20.0	5.2	18	0.44	87.0	2.15	480
NP-60	15.6	5.5	5	0.66	87.7	2.39	629

It is evident from the data in Table 5.9 that the compressive strength of NP cement concrete specimens decreased with an increase in the NP content. The maximum compressive strength was recorded in 50% NP cement concrete (i.e., 20 MPa). The reduction in strength may be due to a reduction in the quantity of cement. This has also resulted in an increase in the water absorption (5.2 and 5.5% for 50 and 60% NP). Similar results were observed in the

time to initiation of reinforcement and I_{corr} that indicate a better performance for the 50% NP concrete than in 60% NP cement concrete (0.44 and 0.66 $\mu A/cm^2$ for 50 and 60% NP). The electrical resistivity of 50 and 60% NP was almost same (i.e., 87 k.Ohm.cm), and both are in range of low risk of reinforcement corrosion. The thermal conductivity of 50% NP cement concrete was much less than those of 60% NP cement concrete (i.e., 2.15 and 2.39 W/mK for 50 and 60% NP). The drying shrinkage increased with an increase in the replacement level of cement with NP. However, the drying shrinkage of 50% NP cement concrete was lower than the threshold value of 500 μm (480 and 629 microns for 50 and 60% NP).

The performance of 50% and 60% NP cement concrete was found to be beneficial. However, optimum performance was observed in concrete with 50% NP, as a replacement of cement, from both the durability and strength perspectives. Such a concrete can be used as a low strength concrete.

Reduction in Greenhouse Gas Emissions from the Production of Cement

Typically, the contribution of CO_2 to global warming is 72% of total greenhouse gases [http://www.timeforchange.org/CO2-cause-of-global-warming]. Hence, it is of great concern that we have taken into consideration the production of 900 kg of CO_2 for every ton of cement production. The production of cement in the Kingdom was 55 million tons. Hence, replacing the cement with industrial waste materials would certainly reduce the generation of CO_2 by same percentage.

CHAPTER 6

CONCLUSIONS, RECOMMENDATIONS AND FUTURE RESEARCH

6.1 Conclusions

This research was conducted to produce medium to low strength concrete utilizing locally available waste materials, such as CKD, LSP, EAFD, OA and NP. Several tests were conducted to assess the mechanical, thermal, and electrical properties of produced concrete. The following conclusions can be drawn based on the data developed in this study:

6.1.1 CKD Cement Concrete Specimens with a w/cm Ratio of 0.45

1. The maximum compressive strength was noted in 5% CKD cement concrete after 28 days of curing. However, the compressive strength development of 5, 10, and 20% CKD cement concretes was not significantly different from that of plain cement concrete.

2. Cement concrete with upto 20% CKD can be utilized as high strength concrete.

3. The water absorption increased marginally with an increase in the quantity of CKD.

4. The corrosion potentials on steel in 20% CKD cement concrete specimens were more than those in the 0% CKD cement concrete. The corrosion potentials on steel

in the concrete specimens with 5 and 10% CKD cement concrete specimens were less than those on 0% CKD cement concrete specimens.

5. The I_{corr} increased with the period of exposure to the chloride solution in all the concrete specimens. The I_{corr} on steel in 5, 10 and 20% CKD cement concrete specimens was less than that on steel in the 0% CKD cement concrete specimens.

6. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. However, the electrical resistivity did not decrease much after a moisture content of 2.5%. All the specimens were in the moderate risk of corrosion based on the electrical resistivity.

7. The thermal conductivity increased with an increase in the quantity of CKD. However, the increase in the thermal conductivity with increasing CKD was not that significant.

8. The drying shrinkage increased with age in all the concrete specimens. Maximum drying shrinkage was noted in the 20% CKD cement concrete. The drying shrinkage in the 5 and 10% CKD cement concrete specimens was less than that in the specimens without CKD.

6.1.2 CKD Cement Concrete Specimens with a w/cm Ratio of 0.55

1. The compressive strength of 50% CKD cement concrete was more than that of 0% and 60% CKD cement concrete after 28 days of curing.

2. Cement concrete with 50 or 60% CKD can be utilized as medium strength concrete.

3. The water absorption increased with an increase in the CKD content. These values in the 50% and 60% CKD cement concrete specimens were more than those in the plain cement concrete.

4. The corrosion potentials were more negative than the ASTM C 876 threshold value of -270 mV SCE in both 50 and 60% cement concrete CKD. Further, the potentials in both the specimens were almost similar.

5. The I_{corr} increased with the period of exposure to the chloride solution in all the specimens. The I_{corr} in 50% CKD concrete specimens was slightly more than that in the 60% CKD cement concrete specimens.

6. The electrical resistivity decreased with an increase in the moisture content. The electrical resistivity at 3% moisture content for 50 and 60% CKD cement concrete was in range of moderate risk of reinforcement corrosion.

7. The thermal conductivity of 60% CKD cement concrete was less than that of 50% CKD cement.

8. The drying shrinkage in 50% CKD cement concrete specimens was less than that in the 60% CKD cement concrete specimens.

6.1.3 LSP Cement Concrete Specimens with a w/cm Ratio of 0.45

1. The compressive strength of 5% LSP cement concrete was maximum. However, the compressive strength decreased with an increase in the quantity of LSP. Further, the

compressive strength of specimens with 10 and 15% LSP was less than that of control specimen.

2. Cement concrete with upto 10% LSP can be utilized as high strength concrete while with 15% LSP can be used as medium strength concrete.

3. The water absorption of concrete increased with an increase in the quantity of LSP. However, the water absorption of 5% LSP concrete specimen was almost similar to that of the control specimen. Further, water absorption of 10 and 15% LSP cement concrete was the same and more than that of 0% LSP cement concrete

4. The corrosion potentials of steel were almost the same in all the specimens. The time to initiation of corrosion was about 58, 75, 15, and 23 days for 0, 5, 10, and 15% LSP cement concretes, respectively.

5. The I_{corr} in all the LSP concrete specimens was less than that on steel in the concrete specimen with 0% LSP, thereby indicating the superior durability of LSP mixtures.

6. The electrical resistivity was almost the same in all the specimens. The electrical resistivity at 3% moisture content in 0, 5, 10 and 15% LSP cement concrete specimens was in the range of low risk of reinforcement corrosion.

7. The thermal conductivity of 5, 10 and 15% LSP cement concrete specimens was almost similar and it was more than that of 0% LSP cement concrete.

8. Maximum drying shrinkage occurred in the 15% LSP cement concrete and minimum in 5% LSP cement concrete. Drying shrinkage specimen with 10% LSP content was a little more than that of 0% LSP cement concrete.

6.1.4 LSP Cement Concrete Specimens with a w/cm Ratio of 0.55

1. The compressive strength decreased with an increase in the LSP content. The compressive strength of 50 and 60% LSP cement concrete was less than that of plain cement concrete. However, the compressive strength of 50% LSP cement concrete was more than that of 60% LSP cement concrete.

2. LSP cement concretes (both 50 and 60%) can be utilized as low strength concrete.

3. The water absorption of 50 and 60% LSP cement concrete was more than that of 0% LSP cement concrete. However, the water absorption of 60% LSP cement concrete was marginally more than that of 50% LSP cement concrete.

4. The corrosion potentials were more negative than the threshold value of -270 mV SCE from the beginning of the exposure in both 50 and 60% LSP cement concrete.

5. The I_{corr} on steel in the 50% LSP cement concrete specimens was more than that on steel in 60% LSP cement concrete.

6. The electrical resistivity decreased with an increase in the quantity of LSP. The electrical resistivity at 3% moisture content for 50 and 60% LSP cement concretes was in the range of negligible to low risk of reinforcement corrosion.

7. The thermal conductivity of 60% LSP cement concrete was less than that of 50% LSP cement concrete.

8. The drying shrinkage of 50% LSP cement concrete was less than that of 60% LSP cement concrete.

6.1.5 EAFD Cement Concrete Specimens with a w/cm Ratio of 0.45

1. Maximum compressive strength was noted in 5% EAFD cement concrete. Further, the strength decreased with an increase in the quantity of EAFD. However, the compressive strength of 10 and 15% EAFD cement concrete was almost similar and less than that of the control mix.

2. The compressive strength of 5% EAFD cement concrete was high while 10 and 15% EAFD cement concrete it was medium.

3. The water absorption of concrete specimens with 5, 10, and 15% EAFD was more than that of plain cement concrete. Further, the water absorption of concrete specimens increased with an increase in the quantity of EAFD.

4. The corrosion potentials on steel in the 15% EAFD cement concrete specimens were more than those in the plain cement concrete specimens resulting in a reduction of the time to initiation of corrosion. The corrosion potentials on steel in 5 and 10% EAFD cement concrete specimens were almost similar and less than that in the plain cement concrete.

5. The I_{corr} increased with the addition of EAFD at early ages. The I_{corr} for 10 and 15% EAFD cement concrete specimens was almost the same. The I_{corr} for 5% EAFD cement concrete was less than that of 10 and 15% EAFD cement concretes. However, the I_{corr} on steel in all EAFD cement concrete specimens was less than that on steel in the plain cement concrete specimens at advanced age.

6. The electrical resistivity was almost the same in the 5 to 15% EAFD cement concrete and was more than that of plain cement concrete. The electrical resistivity at 3% moisture content for all EAFD cement concrete was in the moderate risk of reinforcement corrosion.

7. The thermal conductivity increased with an increase in the quantity of EAFD. However, the increase in thermal conductivity with an increase in the quantity of EAFD was not that significant.

8. The drying shrinkage strain in the 15% EAFD cement concrete content was more than that in 0, 5 and 10% EAFD cement concrete. The drying shrinkage strain in 5% EAFD cement concrete was less than that of plain cement concrete specimens.

9. The initial and final setting time of all the EAFD cement concrete specimens was more than that specified by ASTM C 150.

10. The leachability of heavy metals in the EAFD was within the allowable limits for toxicity characteristics. Hence, all EAFD specimens can be classified as non-hazardous.

11. The performance of all EAFD cement concrete specimens was found to be beneficial. However, the optimum performance from mechanical and durability and thermal perspectives was noted in 5% EAFD cement concrete.

6.1.6 EAFD Cement Concrete Specimens with a w/cm Ratio of 0.55

1. The compressive strength of 0% EAFD cement concrete was more than that of 50 and 60% thermally treated EAFD. However, the compressive strength of 50% EAFD cement concrete specimens was more than that of 60% EAFD cement concrete specimens.

2. The compressive strength of 50 and 60% EAFD cement concrete was in the low strength category.

3. The water absorption of 50 and 60% EAFD cement concrete was more than that of 0% EAFD cement concrete. However, the water absorption of 50 and 60% EAFD cement concrete was almost similar.

4. The corrosion potentials on steel in 50% EAFD cement concrete were slightly more than those in 60% EAFD cement concrete. However, the corrosion potentials in both the concretes were more than threshold value of -270 mV SCE from the beginning of exposure.

5. I_{corr} for 50% EAFD cement concrete was more than that of 60% EAFD cement concrete.

6. The electrical resistivity decreased with an increase in the moisture content in all the concrete specimens. The electrical resistivity at 3% moisture content for 50 and 60% EAFD cement concrete was in range of moderate risk of reinforcement corrosion.

7. The thermal conductivity decreased slightly with an increase in the quantity of EAFD.

8. The drying shrinkage strain of 60% EAFD cement concrete was more than that of 50% EAFD cement concrete.

9. The incorporation of thermally treated EAFD decreased the setting time of concrete as compared to untreated EAFD. However, it also exceeded the allowable limit specified by ASTM C 150 for initial and final setting time.

10. Leachability of heavy metals was within the allowable limits for toxicity characteristics. Hence, all EAFD concrete specimens can be classified as non-hazardous.

6.1.7 OA Cement Concrete Specimens with a w/cm Ratio of 0.45

1. The maximum compressive strength was noted in 5% OA cement concrete. Further, it decreased with an increase in the OA content in the concrete. However, the compressive strength of 10% OA cement concrete was less than that of the control mix.

2. Cement concrete with 5% OA can be utilized as high strength concrete while 10% OA cement concrete can be used as medium strength concrete.

3. The water absorption increased with an increase in the quantity of OA. The water absorption of 10% OA cement concrete was much more than that of 0 and 5% OA cement concretes.

4. The corrosion potentials on steel in the 5 and 10% OA cement concretes were almost similar and more than that in the plain cement concrete. Hence, the time to initiation of corrosion in OA cement concretes was much less than that in the plain cement concrete.

5. The I_{corr} on steel in 5 and 10% OA cement concretes was less than that in the control mixture. However, the I_{corr} on steel in 10% OA cement concrete was slightly more than that in 5% OA cement concrete.

6. The electrical resistivity of 5 and 10% OA cement concretes was more than that of plain cement concrete. The risk of reinforcement corrosion was low in all the OA cement concretes at a moisture content of 3%.

7. The thermal conductivity increased up to 5% OA content and then reduced with an increase in the OA content.

8. The drying shrinkage strain in the 5 and 10% OA cement concrete was slightly less than that of plain concrete specimen. Further, the drying shrinkage strain in 5 and 10% OA cement concretes was almost similar.

9. The concentration of heavy metals were well below the allowable limits. Hence, the OA cement concrete specimens can be classified as non-hazardous.

6.1.8 NP Cement Concrete Specimens with a w/cm Ratio of 0.45

1. The compressive strength decreased with an increase in the quantity of NP in concrete at an age of 28 days. The minimum strength was recorded in the 20% NP cement concrete. However, the compressive strength of 10% NP cement concrete exceeded the strength of control specimen at an age of 90 days.

2. Cement concrete with up to 10% NP can be utilized as high strength concrete with 20% NP can be utilized as medium strength concrete.

3. The water absorption of NP cement concrete specimens was more than that of plain cement concrete specimens.

4. After 230 days, the corrosion potentials were almost the same in all the concrete specimens. The performance of the mix with 20% NP was better than other mixes in delaying the on-set of corrosion.

5. The I_{corr} values in the NP cement concretes were significantly lower than those in the plain cement concrete. The lowest I_{corr} was noted in 10% NP cement concrete while it was almost the same in 15 and 20% NP cement concretes.

6. The electrical resistivity was almost the same in 0, 10 and 20% NP cement concretes. Higher electrical resistivity was measured in the 15% and 20% NP cement concretes. The risk of corrosion in 0, 10, and 20% NP cement concrete was moderate while it was low in the 15% NP cement concrete.

7. The thermal conductivity decreased with an increase in the quantity of NP. However, the variation was not that significant.

8. The drying shrinkage strain in the NP cement concrete was less than that in the plain cement concrete. Minimum drying shrinkage strain was noted in the 10% NP cement concrete. The drying shrinkage strain in 15, and 20% NP cement concretes was found to be almost similar.

9. All NP cement concretes were beneficial. However, the optimum performance from mechanical, durability and thermal perspectives was noted in 10% NP cement concrete.

6.1.9 NP Cement Concrete Specimens with a w/cm Ratio of 0.55

1. The compressive strength of 0% NP cement concrete was more than that of 50 and 60% NP cement concrete. The compressive strength of 50% NP cement concrete was more than that of 60% NP cement concrete. However, the difference in strength was not that significant.

2. Concrete with 50 or 60% NP can be used as low strength concrete.

3. The water absorption increased with an increase in the quantity of NP. However, the water absorption of 50% NP cement concrete was less than that of 60% NP cement concrete.

4. Corrosion initiation in 50% NP cement concrete has shown better performance than 60% NP. The potentials were more negative than the threshold value from the beginning of exposure in specimen with 60% NP.

5. The I_{corr} on steel in 60% NP cement concrete was more than that on 50% NP cement concrete.

6. The risk of corrosion at 3% moisture content in the 50 and 60% NP cement concrete was low, based on electrical resistivity values.

7. The thermal conductivity of 60% NP cement concrete was more than that of 50% NP cement concrete. However, these values were within the ACI range.

8. The drying shrinkage strain in the 50% NP cement concrete was less than that in the 60% NP cement concrete.

9. Both 50 and 60% NP cement concretes were found to be beneficial. However, the optimum performance from mechanical, durability and thermal perspectives was noted in the 50% NP cement concrete.

6.2 Recommendations

The Table 6.1 summarises the avenues for utilization of the developed concrete with the selected local waste materials.

Concrete	Applications
CKD 0 – 20%	High strength structural concrete with moderate durability for beams, columns, footings, slabs, etc.,
CKD 50 – 60%	Non-structural concrete with moderate durability as blinding concrete, pavements, insulation material, trench fill, etc.,
LSP 5 - 10%	High strength structural concrete with high durability for beams, columns, footings, slab, etc.,
LSP 15%	Medium strength structural concrete with moderate durability requirements for beams, columns, slab, footings, etc.,
LSP 50 - 60%	Non-structural concrete with moderate durability as blinding concrete, electric trench material, etc.,
EAFD 5%	High strength structural concrete with high durability for beams, columns, footings, slab, etc.,
EAFD 10 - 15%	Medium strength structural concrete with moderate durability for beams, columns, slabs, footings, etc.,
EAFD 50 - 60%	Non-structural concrete with low durability for blinding concrete, pavements, and blocks etc.,
OA 5%	High strength structural concrete with high durability for beams, columns, footings, slab, etc.,
OA 10%	Medium strength structural with moderate durability applications for beams, columns, slab, footings, etc.,
NP 10%	High strength structural concrete with high durability for beams, columns, footings, slab, etc.,
NP 15 -20%	Medium strength structural applications with moderate durability for beams, columns, footings and slab etc.,
NP 50 and 60%	Non-structural strength concrete with low durability for blocks, blinding concrete, pavements, etc.,

6.3 Future Research

Following are the recommendations for future research.

- Long-term data for a better prediction of durability of concrete with waste materials.
- Evaluate the performance of concrete with waste materials under sulphate-chloride environment, such as sabkha.
- Production of ternary and quaternary cement concretes utilizing the waste materials.
- Exploring the possibilities of using EAFD as a retarder.

CHAPTER 7

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